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Works Construction Paper No. 32

TOLL HIGHWAYS: THEIR ECONOMICS AND CONSTRUCTION

by

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SYNOPSIS

The purpose of this Paper is to demonstrate the possibility of building some of the badly needed long-span road bridges and tunnels in Great Britain as financially self-liquidating projects. The methods of forecasting the diverted and generated traffic to be anticipated on a new crossing are described, and a typical example is given in detail showing how a long-span highway bridge could be financed without resort to public funds, and would pay for itself in a short term of years by means of tolls charged on traffic

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using it. Thereafter it could be freed. Particulars of the operation and excellent financial position of the Mersey tunnel are described.

Numerous examples are also given of toll bridges and tunnels in America and elsewhere, and particulars of some badly needed facilities, such as the proposed Severn, Forth, and Humber bridges, and the Tyne and Dartford tunnels in Great Britain.

The road section of the Paper describes the principles on which the systems of toll roads must be established to be successful, the constitution of the turnpike authority, and its investigations into traffic engineering and revenue, the relation of the full-road system and the turnpikes, and the basis of both in a national highway programme. An account is given of some of the major American turnpikes with an analysis of their uses and sources of revenue.

Part 1

TOLL ROADS

by

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Introduction

The essential unsuitability of the road system in Great Britain for fast motor traffic arises largely from its antiquity. The Romans built many roads which were, in general, remarkably straight, but locally these, and many other roads subsequently built, followed the horse-wagon. Whenever a natural obstacle occurred, the tendency was to circle it, and distance was always subordinate to the necessity of avoiding major cuttings, embankments, and bridges. Sharp curves were of no consequence because speeds were low. A similar state of affairs, in a much more acute form, may be seen in undeveloped territories, where many roads follow the pack mule.

Roads built on these general lines presented no particular difficulty even up to the period when the stage-coach was in its most highly-developed form and speeds between 10 and 20 m.p.h. were achieved, but at about the same time the advent of railways was detrimental to our roads from the point of view of present-day requirements. Throughout the nineteenth century railway promoters were given wide powers, whilst road traffic still relied upon the horse. The result was that at every crossing—and many crossings were at an acute angle—it was the road system which had to give way and the road was first turned, sometimes almost at right-angles in order to cross the railway squarely, and was then turned back on its original line. Nowadays if a railway crosses a road at an acute angle, the expense of a skew bridge has to be accepted. In those days there was no restraint, and a large number of these dangerous crossings still exist even on our important roads.

There are numerous other absurdities which detract from our trunk roads as a means of fast communication. For example, a road will frequently sweep straight up to the entrance gates of some large country seat and will then turn at right-angles and grope its way round the boundary walls. These disadvantages, together with the fact that trunk roads often pass through a rapid succession of congested towns, make fast road communication impossible.

It is clear that a serious factor in the present unsatisfactory situation is the use of the same road by both slow commercial vehicles and traffic capable of higher speeds. Overtaking is made difficult because of the winding nature of the roads

nd the constant frequency of points at which the driver cannot get a clear line of ght ahead.

On most of our main roads, particularly in hilly country, an observer from the air ould see numbers of small convoys each preceded by a slow commercial vehicle vorst of all a large box van) travelling at only about 20 to 30 m.p.h. Behind this could e seen a succession of potentially faster vehicles, possibly six, eight, or even more, aiting their chance to overtake, and as often as not being delayed by the driver of ome car of low horsepower or poor mechanical condition who will not venture to vertake until he can see the road for a considerable distance ahead. Worse still iere may be a second slow and heavy vehicle which is only capable of overtaking at relative speed of 1 or 2 m.p.h. and therefore requires a considerable distance in hich to do so. Such heavy vehicles always seem to follow each other as closely as ossible, so that there is no space between them for faster traffic to overtake them ie at a time. In the face of this frustration, a great strain is thrown on the self-ntrol of any driver; every now and then some individual reaches the end of his itience and takes a chance, which is a fruitful cause of accidents.

Even our trunk roads consist, for the greater part, of only three traffic lanes. his means that overtaking vehicles from either direction are incessantly contesting ie use of the centre lane. This lane is inevitably narrowed by the fact that the ivers of slower traffic do not care to keep well to their near side but normally aintain a distance of about 6 ft from the verge. This practice is understandable drivers of heavy vehicles since many of our roads are over-cambered, and they iturally tend towards the crown of the road for this reason.

Improvements which take the form of widening the original road and smoothing t sharp curves, as well as substituting hedges by open fences to improve sight lines, n only be palliative. By-pass roads around towns and cities are often of more -to-date construction and are more valuable. Unless special precautions are ken, however, they invite an outcrop of dwelling-houses, factories, and public-uses which, as has often occurred in past years, choke these roads.

It is true to say that there has been no comprehensive construction of a vital stem of highways in Great Britain since the Romans laid the basic pattern for their n requirements. British Governments have never provided an adequate road twork.

The only type of road which is suitable for present-day traffic is that consisting of al carriageways, each of at least two unit lanes 12 ft in width, the carriageways ing separated by plantations of shrubs that act as a headlight screen both winter d summer.

The total length of this type of highway in the whole of the British Isles is absurdly all.

The most pressing needs of our highway system today are:

(*a*) New high-speed trunk roads.
(*b*) Improvements of existing roads.

Our existing trunk roads, although adequate for the intercommunication of rural l urban communities, are quite useless for the purpose of interconnexion of our in industrial centres, towns, and docks.

n 1938, the County Surveyors' Society made proposals for about 1,000 miles of torways, and the need has grown since then, though very little has been done.

Requirements

The requirements for the construction of an adequate road system are:

(1) Finance
(2) Manpower
(3) Plant
(4) Materials
(5) Technical knowledge
(6) Traffic estimates
(7) Reduction of legal delays to the minimum.

1. *Finance.* The one requirement which has precluded the development of our highway system in the past and continues to do so today is money.

(*a*) *Present finance.* Much has been written about the financing of our roads at the present time. Useful recent resumés were given by the British Delegation at the Tenth Congress[1] of the Permanent International Association of Road Congresses at Istanbul, and by Proudlove[2] before the British Association.

The essential point which must be faced in the present financing of our roads is that the Road Fund is now only a name. Motor vehicle and fuel taxes are paid direct into the Exchequer as revenue, and our trunk roads are largely the financial responsibility of the Government. Class I, II, and III roads are financed by the Government to the extent of about 75%, 60%, and 50% respectively as funds are available. Since the war £2,286 millions have been collected in taxes associated with motor vehicles and £264 millions spent on roads. It has been announced that the Government is going to allocate £147 millions to road construction in the next 4 years, and some of this expenditure has already been authorized but it appears that some years will pass before works begin.

It is increasingly evident that it is quite impracticable to provide out of Government funds the highway system which we so vitally need, particularly now that there is a movement to curtail Government spending.

We are not alone in facing this problem. A report submitted to Congress in the United States in 1955 by the Secretary of Commerce has this to say about finance:

> "It has become increasingly apparent that public funds applicable to highway construction, especially on the more important routes, have not been sufficient to prevent their increasing obsolescence. On the contrary the conclusion is clear that the condition of the more important highways is falling steadily further behind the needs of traffic. It has likewise become more apparent that the gap will not be closed by the use of current revenues on a pay-as-you-go basis at the present rate of taxation. Although tax rates and other fees have been raised to some extent, the increases have not even offset the higher road costs resulting from wartime and postwar inflation."

If the United States of America cannot finance their highways from central government resources, how much more is this the case in Britain?

(*b*) *Finance under a toll system.*—A trunk road having dual carriageways each of 32 ft width costs about £300,000 per mile.

There are several ways of raising the necessary capital for constructing a motorway, and most of these methods are applied in the United States. In some cases bonds

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[1] The references are given on p. 610.

re issued by a Turnpike Commission and backed by the Federal Government or he State. These are taken up sometimes by public subscription, sometimes by egotiation with finance corporations, the banks, or insurance companies. Some of he bonds are redeemable, some are revenue bonds. Bonds have been issued with o other backing than the prospective revenue from the tolls to be charged. Often combination of these sources of capital is adopted.

Those observers who, although reluctant to give wholehearted support to the toll ystem, are inclined to support a limited application of it, usually qualify their cceptance with the stipulation that the toll highways should finally revert to public wnership and the tolls be removed.

It is probable that redeemable bonds are the type most likely to be palatable to ne British public.

Generally the aspect of proposed toll highways of most interest to the user is ne extent of the toll likely to be charged. In the United States this varies, but is bout 1 cent per mile for motor cars, 2 cents per mile for public transport vehicles nd 2½ cents per mile for commercial vehicles.

. *Manpower.* There has been much discussion recently in the national and technical ress regarding our ability to provide sufficient manpower to carry out an extensive oad-construction programme without detriment to other priorities, particularly nose linked with our exports. Perhaps the most significant observation in this onnexion was made by Sir George Mowlem Burt in a recent article.[3] He stated Some have expressed doubt about our industry's ability to cope with the vast rogramme that should be introduced, without harming some other priority. It is orth reminding them that the vital need is for 700 miles of new dual-carriageways: ne airfield runway programme undertaken by us during the war was the equivalent f 3,000 miles of dual-carriageways. Civil engineering contractors—backed by the ighest degree of mechanisation in Europe—have ample resources of skilled men and odern plant available to cope with a full-scale road programme—and that with o question of sacrificing other priorities."

The labour force required for an extensive road-construction programme would be omparatively small now that much of the work to be done is mechanized.

Plant. There is already in Great Britain adequate specialized plant to commence erations on the building of new highways together with the necessary qualified ant drivers. In addition, our future requirements of plant can now be met by ritish manufacturers.

Materials. By far the greatest quantity of material required for the construction a satisfactory system of highways is concrete. There are adequate supplies of itable sand and stone for this purpose and the cement requirement would be about 200 tons/mile of dual carriageway each of 24 ft width.

This does not, of course, allow for ancillary works. It is estimated that the total quirement of cement for 1,000 miles of toll highways would be about 1,500,000 tons.

Technical knowledge. We have the technical knowledge in Great Britain in all the rious aspects of modern road building to carry out the design and construction of adequate system of highways. Our road engineers have prepared innumerable hemes for proposed highways, and have given a great deal of careful thought to em. They have not so far, however, been fortunate enough to have seen many of

their schemes carried out. In the specialized branches there are international experts in site investigation, soil stabilization, concrete-quality control, and many other fields, and our contracting organizations have the capacity to get on with the work.

6. *Traffic estimates.* Much time has been spent by various organizations in recording and analysing traffic densities. For a toll highway a reliable estimate of the traffic which is likely to use the road is a first necessity. It is interesting in this connexion to note that the traffic estimate for the New Jersey Turnpike in the United States upon which the project was based has been exceeded two-fold.

7. *Reduction to a minimum of legal delays.* Apart from the urgency of the problem and the financial considerations which must be settled before the highways can be started, there is the question of speed of construction. It is essential, if highways are to be at an economic cost, that they be constructed with expedition. Works are often held up because of legal complications. A means must be found to avoid legal delays, which can upset the most carefully prepared programme.

Part 2

TOLL BRIDGES AND TUNNELS

by

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Introduction

The purpose of this Paper is to demonstrate the possibility of building some of th badly needed long-span road bridges and tunnels in Great Britain as financiall self-liquidating projects. The necessity for doing so is obvious, unless indeed w are prepared to continue to do without them indefinitely, for there seems to be littl prospect of our being able to afford to build them out of public funds.

No long-span bridge has been built in Great Britain since the Forth Bridge wa completed 65 years ago. No long-span roadway bridge has ever been built her And yet, owing to the growth of motor traffic in the past 40 years, no less than twent nine such bridges, with spans varying from 1,000 to 4,200 ft, have been built i America. How could they afford it? The answer is simply that the bridges hav been built without exception as self-liquidating projects.

As regards tunnels in the United Kingdom we are slightly better served. Tl Blackwall and Rotherhithe tunnels are free; the Mersey tunnel, for which tolls a charged, has proved to be an outstanding financial success.

Particulars of the financing of a number of bridges and tunnels, mostly in tl United States, will be found later in this Paper; they represent an extraordina record of engineering skill backed by business acumen. The majority have pa their way handsomely from the start and none of them has got into financial dif culties. It is hard to see any reason why the same thing could not be done here.

As stated in the first part of the Paper, the essential requirements are engineering nowledge, materials, manpower, plant, and money. Considering these individually, iere is no question but that British engineers have the knowledge both for the esign and the construction. The designs for the Forth, Humber, and Severn adway bridges and for the Dartford and Tyne tunnels have already been made id there is no lack of experienced British contractors to undertake the work.

There is little difficulty as regards materials. Not more than 25,000 tons of steel-ork would be required for any of the bridges; this could be spread over a con-ruction period of (say) 5 years and would amount to only 5,000 tons per annum. his is much less than 1% of the annual structural steel output of the country. In ddition about 7,500 tons of high-tensile wire would be required for the cables of the evern or Forth bridges and 12,500 tons for the Humber bridge; this might well be ie material in shortest supply.

As regards manpower, this already exists in the fabricating shops, and for the site ork not more than 300 to 400 men would be needed. Mechanization has consider-bly reduced the labour force required. All the bridge-building plant, with the ossible exception of the cable-spinning machines, is available. The supply of aterials for tunnels presents no difficulties; for example, the proposed Dartford innel will use a total of 35,000 tons of cast-iron segments during the first 3 years of onstruction, a figure which would cause no strain on the special foundries equipped r this work. Other materials, such as cement and steel, are only required in omparatively small quantities.

Most of the contractors who specialize in tunnel work have available some of the ecial plant required for tunnelling; indeed, at Dartford most of the plant is already stalled.

The manpower problem is rather more difficult than in the case of bridges. The ck of tunnel work during the last generation has reduced reserves of skilled tunnel-ig labour to a low level.

Finally, the money can be obtained by the issue to the public of bonds, the interest i which is secured by the tolls to be charged on traffic using the bridge. Full etails of figures showing how the financing of a typical self-liquidating project ould be carried out in Great Britain are given later in this Paper.

This may be an opportune place to deal with the natural objection that anyone is to paying a toll:—

(1) No motorist need use the bridge or tunnel and pay the toll unless he wishes. e can still, if he prefers, make the detour he had to make before the new crossing is provided.

(2) Many years of frustration have shown that the choice is not between a toll ossing and a free one. It has proved to be between a toll crossing and no crossing all.

FORECAST OF TRAFFIC LIKELY TO USE A PROPOSED CROSSING[4]

The traffic using the new crossing will be made up of traffic "diverted" from existing ossings and "generated" traffic, which is traffic attracted by reason of the conven-ice of the new crossing. Due allowance should be made for normal traffic growth each case.

The study of road maps, supplemented by test runs, will indicate routes from which verted traffic is likely to be drawn.

Directional traffic surveys[5] should be undertaken at all existing crossings on these

routes. In addition, simpler censuses of total traffic on the approach roads should be taken over a 12-month period to show the hourly, daily, and monthly variations. Preliminary information of this nature can be obtained from the Ministry of Transport Road Traffic census.[6]

The proportion of traffic diverted depends upon mileage, journey time, tolls, physical and psychological characteristics such as types of highway, scenery, etc. These factors may best be reduced to a common basis of monetary value, including obvious costs such as tolls, fuel, and driver's time, and hidden costs such as wear on tires, depreciation, and maintenance, etc.

From these considerations the "minimum" and "probable" traffic likely to be diverted can be estimated. Allowance must be made for the increase owing to the normal rate of growth of traffic by the date the crossing will be opened. In Great Britain today this is $5\frac{1}{2}\%$ per annum.

The first year's traffic is always found to be greater than the estimated diverted traffic. The surplus is known as the "generated" traffic. Table 1 gives the generated traffic during the first year on a number of actual toll crossings, expressed as a percentage of the diverted traffic.

TABLE 1

Crossing	Opening year	Generated traffic as % of diverted traffic
In Great Britain		
Mersey tunnel	1934	145
In New York		
Holland tunnel	1927	125
Goethals bridge	1928	187
Outerbridge	1928	58
George Washington bridge . .	1931	65
Bayonne bridge	1931	244
Triborough bridge	1936	105
Lincoln tunnel	1937	195
Elsewhere in U.S.A.		
Philadelphia–Camden bridge .	1926	131
Carquinez bridge	1927	53
Tacony–Palmyra bridge . . .	1929	234
San Francisco–Oakland bridge .	1936	239
Golden Gate bridge	1937	
Tacoma Narrows 1st bridge . .	1940	140
,, ,, 2nd bridge .	1950	96

The figures for Tacoma Narrows bridge are taken from data kindly supplied by the Washington Toll Bridge Authority. The remaining figures are taken from reference 4.

It will be seen that the values vary from 53% to 244% with an average of 144%. Even in the United States, where there is considerable experience in traffic forecasting, it is rare to assume a value of more than 50% for the estimate of minimum traffic. A considerable degree of judgement is called for in deciding the percentage to allow for "generated" traffic and in doing so the following points should be borne in mind:

Fig. 2.—The Mersey Tunnel, 17 July, 1934: completed 44-ft. internal-dia. tunnel under river

Fig. 3.—Old Haymarket entrance and Plaza, Liverpool

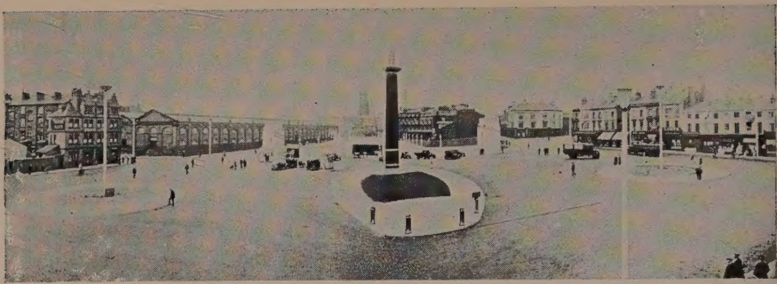

Fig. 4.—Chester Street entrance, Birkenhead

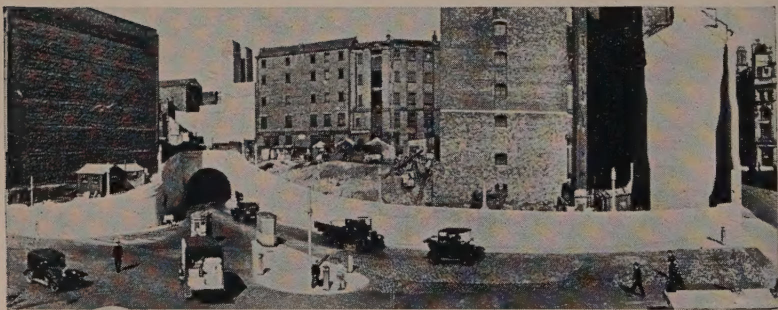

Fig. 5.—New Quay entrance

Fig. 6.—Proposed Severn bridge

Consulting Engineers: Mott, Hay & Anderson and Freeman, Fox & Partners.
Clients: Ministry of Transport and Civil Aviation.

Fig. 7.—Proposed Humber bridge

Consulting Engineers: Freeman, Fox & Partners. Clients: City & County of Kingston-upon-Hull.

(1) Short-journey traffic is more likely to be generated than long-journey traffic.

(2) Crossings in or near large cities are likely to generate a higher percentage than those on trunk roads out in the country.

(3) Crossings which cut out a long detour and replace poor ferry services are likely to generate a high percentage.

(4) Crossings which connect large cities with relatively undeveloped areas are likely to generate a high percentage but the generated traffic may be spread over several years.

There are few proposed major crossings in Britain where it would be wise to allow more than 50% for the minimum estimate of generated traffic, but few where the probable value is less than 100%.

Fig. 1 shows the Ministry of Transport traffic index, from which it can be seen that over a long period, due to the incidence of war and economic depression, etc., the minimum long-term annual rate of growth of traffic is from 2 to $2\frac{1}{2}$% per annum.

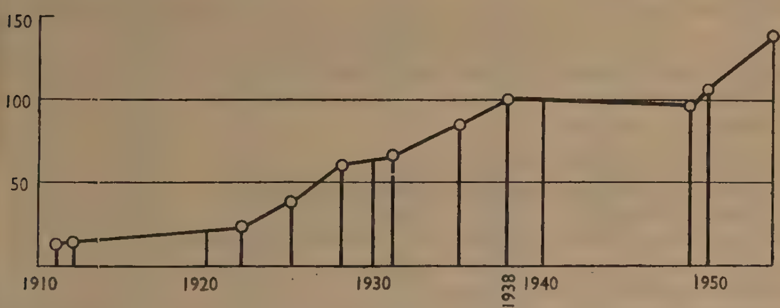

Fig. 1.—Traffic index

Average daily traffic at 191 points on trunk and class 1 roads for 7 consecutive days in August between 6 a.m. and 10 p.m. The 1938 figure (931,000 motor vehicles) is taken as 100. In this Figure horse-drawn traffic has been added to the M.T. traffic index for mechanically propelled vehicles.

Toll-collection facilities

The usual practice is to build a toll plaza, in which the traffic lanes fan out, in the approach road at one end of the bridge. At least two toll booths are provided for each traffic lane and it is found that cars can be passed at an average rate of one every 5 or 6 sec, at each toll booth, though it is wise to allow 10 sec per vehicle. By this means there is normally no appreciable delay and even at peak periods there is usually less delay than is frequently experienced at traffic-lights.

The peak-hour traffic is generally greater in one direction than in the other (normally about two-thirds of the total traffic is in one direction). Therefore the lanes past the inner booths should be reversible. Tolls should always be collected on the driver's side. A canopy should be provided over the toll booths, which should be as narrow as possible. The outside lanes at the toll booths should make provision for extra-wide loads.

The toll plaza should be sited in such a position that additional lanes and toll booths can be added as they become necessary. A small administration building

should be provided near the toll booths and should include cloakroom facilities for the collectors. Toll plazas should be approximately level so that vehicles do not tend to run back while waiting their turn at the booths.

Data on the design of plazas are provided in reference 7.

The economic aspect of toll crossings

The promotion, building, and operation of a toll crossing is probably best done by a Bridge Authority established by an appropriate Act of Parliament. The necessary capital is raised by the issue to the public of bonds. If the capital and interest are guaranteed by the Government or by Local Authorities the rate of interest that has to be offered will, of course, be substantially less than if the issue is simply of revenue bonds, which rely solely for their security on the proceeds from the tolls. It has been found in the United States that bonds which can be repaid as and when funds are available are preferable to those which are repayable on a fixed date. These latter bonds require the establishment of a sinking fund on which the interest is generally less than that on the loan.

The economic aspect is best illustrated by a hypothetical example which is fairly typical of the proposed Humber, Severn, or Forth bridges:

Estimated traffic (allowing for effect of toll) in first year:
Minimum 2,000,000 mechanically propelled vehicles/year
Probable 3,000,000 ,, ,, ,, ,,

Estimated rate of growth of traffic:
Minimum 40,000 mechanically propelled vehicles/year
Probable 150,000 ,, ,, ,, ,,

Estimated average saving in milage: 15 miles } for diverted traffic
,, ,, ,, ,, journey time: 40 min }

Estimated construction period: 5 years

Assume a loan of £16,500,000 @ 4½% repayable as funds become available (or with sinking fund @ 4½%).

Amount of loan is derived as follows:—

Construction cost (for free bridge)		£12,000,000
Toll facilities		30,000
Administrative, engineering, and legal costs		900,000
Interest during construction	£3,720,000	
less re-investment (say)	930,000	
		2,790,000
Contingencies reserve		780,000
	Total loan:	£16,500,000

Estimated annual operation and maintenance costs:

Administration and toll collection	£40,000
Maintenance	50,000
Insurance	20,000
	£110,000 per annum

Assume maintenance costs are halved during the first 2 years, giving total operating and maintenance cost of £85,000.

Assume the lowest tolls which would leave a small surplus after paying interest,

operating, and maintenance costs in the first year, assuming the minimum estimated traffic. This is an average of 8*s* 6*d* on mechanically-propelled vehicles (i.e., excluding cycles).

The annual state of the finances of the bridge authority is shown in Tables 2 and 3.

TABLE 2

Year	Minimum traffic					
	Annual traffic × 1,000	Gross income × £1,000	Net income × £1,000	Interest × £1,000	Available for repayment × £1,000	Loan outstanding × £1,000
0	—	—	—	—	—	16,500
1	2,000	850	765	743	22	16,478
2	2,040	867	782	742	40	16,438
3	2,080	884	774	740	34	16,404
4	2,120	902	792	739	53	16,351
5	2,160	919	809	736	73	16,278
6	2,200	936	826	732	94	16,184
7	2,240	953	843	728	125	16,059
8	2,280	970	860	723	137	15,922
9	2,320	987	877	717	160	15,762
10	2,360	1,004	894	709	185	15,577
11	2,400	1,021	911	701	210	15,367
12	2,440	1,038	928	691	237	15,130
13	2,480	1,055	945	682	263	14,867
14	2,520	1,072	962	669	293	14,574
15	2,560	1,089	979	656	323	14,251
16	2,600	1,106	996	641	355	13,896
17	2,640	1,123	1,013	626	387	13,509
18	2,680	1,140	1,030	609	421	13,088
19	2,720	1,157	1,047	589	458	12,630
20	2,760	1,174	1,064	569	495	12,135
21	2,800	1,191	1,081	547	534	11,601
22	2,840	1,208	1,098	522	576	11,025
23	2,880	1,225	1,115	497	618	10,407
24	2,920	1,242	1,132	468	664	9,743
25	2,960	1,259	1,149	438	711	9,032
26	3,000	1,276	1,166	407	759	8,273
27	3,040	1,293	1,183	372	811	7,462
28	3,080	1,310	1,200	336	864	6,598
29	3,120	1,327	1,217	297	920	5,678
30	3,160	1,344	1,234	255	979	4,699
31	3,200	1,361	1,251	212	1,039	3,660
32	3,240	1,378	1,268	165	1,103	2,557
33	3,280	1,395	1,285	115	1,170	1,387
34	3,320	1,412	1,302	62	1,240	147
35	3,360	1,429	1,319	7	1,312	—

Repaid in 35 years

It will be seen that if the traffic proved to be the estimated minimum, the loan would be paid off in 35 years and if the estimated probable, in 15 years. The bridge could then be freed of toll and handed over to the Ministry of Transport for maintenance.

TABLE 3

Year	Probable traffic					
	Annual traffic × 1,000	Gross income × £1,000	Net income × £1,000	Interest × £1,000	Available for repayment × £1,000	Loan outstanding × £1,000
0	—	—	—	—	—	16,500
1	3,000	1,275	1,190	743	447	16,053
2	3,150	1,338	1,253	723	530	15,523
3	3,300	1,403	1,293	700	593	14,930
4	3,450	1,467	1,357	673	684	14,246
5	3,600	1,530	1,420	641	779	13,467
6	3,750	1,594	1,484	606	878	12,589
7	3,900	1,657	1,547	567	980	11,609
8	4,050	1,721	1,611	523	1,088	10,521
9	4,200	1,785	1,675	474	1,201	9,320
10	4,350	1,849	1,739	419	1,320	8,000
11	4,500	1,912	1,802	360	1,442	6,558
12	4,650	1,976	1,866	295	1,571	4,987
13	4,800	2,040	1,930	224	1,706	3,281
14	4,950	2,104	1,994	148	1,846	1,435
15	5,100	2,167	2,057	65	1,992	—

Repaid in 15 years.

In practice, when it was seen that the project was paying off so well, tolls would probably be reduced and the project re-financed by a shorter-term loan at lower interest.

If the traffic proved to be the estimated probable volume, it would be possible, after 2 or 3 years' operation, to re-finance the project with a shorter-term loan at a lower rate of interest and reduce the tolls.

It should be noted that if a loan could be raised at $3\frac{1}{2}\%$ instead of $4\frac{1}{2}\%$, the average toll could be 6*s* 6*d* and the loan would be paid off in 40 years with "minimum" traffic, or 18 years with "probable" traffic.

An average toll of 8*s* 6*d* could be obtained by the typical toll schedule in Table 4, in which some idea of the cost of avoiding the bridge is also given:

TABLE 4

Vehicle	% of total of mechanically propelled vehicles	Typical toll	Cost of detour				Total cost of detour	Equivalent saving in operation of vehicle
			15 miles		40 min			
			per mile	total	per hour	total		
Heavy goods . .	25	14*s*	1*s*	15*s*	3*s* 6*d*	2*s* 4*d*	17*s* 4*d*	3*s* 4*d*
Light goods . .	12	8*s*	8*d*	10*s*	3*s*	2*s*	12*s*	4*s*
Buses and coaches	6	13*s* 6*d*	1*s*	15*s*	5*s*	3*s* 4*d*	18*s* 4*d*	4*s* 10*d*
Cars	51	6*s*	6*d*	7*s* 6*d*	2*s* 6*d*	1*s* 8*d*	9*s* 2*d*	3*s* 2*d*
Motor-cycles . .	6	2*s*	2*d*	2*s* 6*d*	2*s*	1*s*	3*s* 6*d*	1*s* 6*d*
Pedal cycles . .	10	6*d*	—	—	1*s* 6*d*	1*s*	1*s*	6*d*

From Table 4 it can be seen that the cost to vehicle operators (excluding pedal cyclists) of the lack of this bridge at present averages 11*s* 9*d* per journey. After paying toll, vehicle operators using the bridge would still save an average of 3*s* 4*d* per journey. This would become the full 11*s* 9*d* when the loan is paid off and the bridge freed of toll.

Assuming that the saving to generated traffic is only half that to diverted traffic and that the percentage of probable generated traffic is 100%, then the probable saving to vehicle operators after paying tolls is:

1,500,000 @ 3*s* 4*d* = £250,000
1,500,000 @ 1*s* 8*d* = £125,000

£375,000 in the first year

The above estimates have been prepared for a long-span highway bridge, but if the necessary modifications were made to allow for the additional upkeep and working costs of a tunnel, they would apply equally to that. A tunnel requires a high standard of ventilation and continuous lighting as well as regular patrolling, cleaning, and pumping, whereas a bridge only requires normal road cleaning, lighting at night, and regular painting. For a tunnel, therefore, the scale of toll charges would have to be increased or the period for repayment of loan extended. Alternatively a tunnel might well be promoted as a partially self-liquidating project, the balance of the construction cost being borne by public funds.

A toll tunnel in the United Kingdom—the Mersey Tunnel

Work on the Mersey tunnel (Fig. 2), connecting Liverpool (Fig. 3) and Birkenhead (Fig. 4), was started on the 16th December, 1925, and the tunnel was opened to traffic on the 18th July, 1934.

It consists of a main tunnel which passes under the River Mersey with two branch tunnels, one at each side of the river with entrances near the docks (see Fig. 5). The main tunnel is 44 ft wide throughout its length, being circular in section under the river and having a flattened invert in the landward sections. The branch tunnels are 26 ft 6 in. wide with flattened inverts.

There is a four-lane carriageway, 36 ft between kerbs, in the main tunnel and the branch tunnels each have a two-lane carriageway 19 ft wide.

Despite the fact that there has been no increase in toll charges, which vary from 1*s* per journey for an 8 h.p. car to 5*s* per journey for a large motor-coach, since the tunnel was opened in 1934, the annual toll revenue is now more than double what was anticipated when the scale of tolls was made.

Table 5 shows that there has been a steady increase in the number of vehicles using the tunnel and that there is now four times as much traffic as in the first year of operation.

The density of traffic at peak hours—8.30 a.m. to 10.0 a.m. and 4.30 p.m. to 6.15 p.m.—is such that it is necessary to close the branch tunnels during these periods in order to preserve the steady flow of vehicles in the main tunnel, which would otherwise be interrupted by traffic to and from the branch tunnels. This peak traffic is very largely one way and it has been found necessary to use three lanes in one direction and one in the other at these times.

It may be of interest to briefly outline the financial arrangements made for the construction and running of the only toll tunnel in Great Britain.

The total capital expenditure on the undertaking, including purchase of land and

TABLE 5.—MERSEY TUNNEL, FOUR LANES

Year	Vehicles per year	Toll revenue	Average per vehicle
1st year of operation (18/7/34–17/7/35)	2,269,665	£166,879	1*s* 5*d*
Year ending 31/3/39	3,872,982	£300,019	1*s* 7*d*
„ „ 31/3/48	5,204,880	£447,594	1*s* 9*d*
„ „ 31/3/49	4,997,725	£456,676	1*s* 10*d*
„ „ 31/3/50	5,594,831	£505,129	1*s* 10*d*
„ „ 31/3/51	6,553,429	£580,575	1*s* 9*d*
„ „ 31/3/52	7,540,002	£640,895	1*s* 8*d*
„ „ 31/3/53	7,998,464	£651,805	1*s* 8*d*
„ „ 31/3/54	8,494,789	—	—
„ „ 31/3/55	9,132,638	—	—

easements, construction, legal and parliamentary expenses, amounted to just over £7,500,000; of this total the Ministry of Transport contributed £2,500,000 and £55,000 was borne by Birkenhead. The remainder was financed out of money borrowed by Liverpool.

The cost of operation and management is met out of funds provided by the Corporations of Liverpool and Birkenhead. These contributions are in proportion to the rateable values of the two authorities but with a statutory limitation of the amount paid by Birkenhead. Expenditure met from this source covers the cost of repair, maintenance, and lighting of the tunnel, police, cleaning and breakdown services, and administration costs.

The Mersey Tunnel Acts provide that after meeting the cost of collection the toll income shall be applied to certain statutory funds of the undertaking including interest on and redemption of the capital expenditure on construction, etc. After these statutory requirements have been met any balance of toll income remaining may be applied to the relief of payments made by Liverpool and Birkenhead. Since 1948 there has been sufficient excess of toll revenue to reimburse the two authorities for the total amount of their contribution towards the operational expenses.

LONG-SPAN BRIDGES REQUIRED IN THE UNITED KINGDOM

From the following particulars it can be seen, by reference to the typical examples worked out above, that there would be no difficulty in building the proposed Severn, Humber, and Forth bridges as financially self-liquidating projects.

1. *The proposed Severn bridge.* It is estimated that the proposed new bridge (Fig. 6) from Beachley to Aust would cost between £10,000,000 and £15,000,000, depending on its capacity and the extent of the new approach roads included in the project. The need for such a bridge to speed the England–Wales traffic has been felt for many years; it would shorten the distance from Bristol to South Wales by 55 miles. The only existing facilities are the car ferry at Beachley, which carries few vehicles and cannot run in rough weather, and the bridge in Gloucester 28 miles upstream. The new bridge would carry not less than four traffic lanes and its span would be about 3,240 ft.

Forecasts of traffic show that the minimum volume likely to use the bridge in its first year would be 2,000,000 vehicles, and the probable traffic about 3,500,000.

2. *The proposed Humber bridge.* The proposed river crossing (Fig. 7) between Barton-on-Humber and Hessle would consist of a suspension bridge of 4,500-ft span. This would be the longest single span in the world, exceeding that of the Golden Gate bridge by 300 ft. Its cost today would not exceed £13,000,000, assuming the roadway carried not less than four traffic lanes.

The lowest bridge over the river is at Boothferry, 28 miles inland from Hull and 50 miles from Grimsby; this involves north–south traffic in an 80-mile detour. The only other crossing is by the car ferry, which carries 20 vehicles and is frequently interrupted in stormy or foggy weather. Besides developing the south side of the Humber, the bridge could well form the centre of an alternative north–south highway, east of the Great North Road.

Estimates of the traffic to be anticipated on the new bridge, based on the road census figures, show that it would be likely to exceed that on the proposed Severn bridge. The figures show that the minimum volume of traffic in the first year would be 2,400,000 vehicles and the probable volume about 3,200,000 vehicles. Within 10 years the traffic would be doubled.

Furthermore it is estimated that not less than 1,000,000 gal of petrol and diesel fuel would be saved per annum.

The total saving to road users amounts to £1,250,000 per year.

3. *The proposed Forth bridge.* The need for a roadway crossing of the Firth of Forth has been felt for many years; hence the variety of schemes put forward, which include road bridges of various spans, modification of the existing railway bridge to enable it to carry roadway traffic, and the construction of a tunnel or subway on the harbour bed. We are only concerned with the feasibility of constructing the Forth suspension bridge as a self-liquidating project.

Until a crossing is provided, traffic will have to continue to use the half-hourly ferry service or make a 30-mile detour inland via the Kincardine bridge.

The proposed new bridge (Fig. 8) is located ½ mile or so to the west of the Forth railway bridge. It has a main span of about 3,300 ft, very much the same as that of the proposed Severn bridge, and the cost (for a bridge to carry not less than four lanes of traffic) would be similar.

In 1947, when the total cost of the new bridge was estimated at £6,200,000, the Ministry of Transport was prepared to make a grant of 75% towards the cost, the balance to be met partly by tolls to be levied for 30 years and partly by the rates, but owing to lack of funds a start on the work was never authorized.

Estimates show that the minimum traffic in the first year would be about 1,700,000 vehicles and the probable traffic about 2,700,000.

Tunnels required in the United Kingdom

1. *Dartford tunnel.* The site of the Dartford–Purfleet tunnel lies 13 miles downstream of the lowest existing permanent crossing of the Thames, the Blackwall tunnel. Ferries operate at Gravesend and Woolwich, 7 miles downstream and 11 miles upstream respectively from the proposed tunnel, which will link up the projected north and south orbital roads and thus fulfil a vital function in highway plans designed to keep traffic out of the built-up areas of London. The tunnel will also greatly improve communications for industry along the banks of the lower Thames and will encourage further development.

Traffic investigations have shown that the ferries are avoided largely at the expense

of additional milage, considerable detours by way of the Blackwall and Rotherhithe tunnels being preferred. One enquiry revealed that some fifty industrial concerns alone estimated between them an annual saving in milage, fuel, and man-hours amounting to £140,000 on completion of the Dartford tunnel.

The estimated cost of the scheme is £10,624,000. The Dartford Tunnel Act of 1930 provided for £3,000,000 to be paid from the Road Fund towards the cost of the tunnel out of the estimated cost (at that time) of £3,500,000. The Kent and Essex County Councils were to contribute jointly £500,000. The Act also empowered the Minister of Transport to make a further grant from the Road Fund in respect of the cost of works exceeding the estimate, and this further sum with interest, if any, is to be repaid out of tolls.

The estimated volume of traffic expected to use the tunnel is 2,080,000 vehicles per annum.

2. *Tyne tunnel.* The site of the proposed tunnel is 6 miles east of Newcastle and 3 miles west of North Shields and South Shields.

On both banks of the Tyne there are heavy-engineering and ship-building works, and in the surrounding districts lie the main coal-fields of the north-east. New light industries have been developed in the area in recent years, and as the labour potential is favourable, these are expected to increase. Within an area 15 miles north and south of the Tyne and between trunk road A.1 and the coast, there is a population of 1,500,000. A further 500,000 people live in the Tees-side area a little to the south.

The only existing means for vehicles crossing the river between Newcastle and the coast is the use of ferries of limited capacity at the site of the proposed tunnel and at North and South Shields. Only one of the bridges at Newcastle–Gateshead is not subject to special load restrictions.

These conditions give rise to intense traffic congestion in Gateshead and Newcastle, to which the new tunnel will afford radical relief. The tunnel will also be a focal point of widespread road development, which must greatly increase the efficiency of industrial communication in the area served.

Information on which the anticipated volume of traffic using the tunnel could be assessed is not available in sufficient detail to offer a reliable guide. An estimate of 5,000 vehicles per day has been put forward, but this is thought to be a cautious figure.

The estimated cost of the tunnel and its approaches is £10,750,000.

Examples of toll bridges in the Commonwealth

1. *Sydney Harbour bridge (1924–32). Arch span 1,650 ft.* The cost of this bridge, exclusive of interest and land charges, was £6,266,000 and the funds for construction were provided by the Government. During the past few years there has been an annual surplus of income over expenditure of the order of £300,000, and this amount is tending to increase. If all this surplus was put to paying off the debt and interest the bridge would be freed within 19 years or less.

There was originally no intention of charging tolls, but after the bridge was completed it was decided, on account of the economic depression, to charge tolls to pay for its maintenance. On account of the volume of traffic the toll charges were originally fixed at an extraordinarily low scale, the charge for a car and driver being 6*d*, with 3*d* extra per passenger. This was much less than the charge on the ferries

Fig. 8.—Proposed Forth bridge

Consulting Engineers: Mott, Hay & Anderson. Clients: Forth Bridge Committee.

FIG. 9.—AUCKLAND HARBOUR BRIDGE

Consulting Engineers: Freeman, Fox & Partners. Clients: The Auckland Harbour Bridge Authority, New Zealand. Contractors: Cleveland Bridge and Dorman Long (Auckland).

It appears that the tolls could have been substantially increased without causing any hardship and the financial position of the bridge thereby improved.

The bridge finances suffered during the second world war, when much of the traffic was military and therefore exempt from payment of toll.

Since the bridge was opened, the volume of traffic on it has increased from 1,000,000 to 15,000,000 vehicles per annum in 1953. The capacity of the bridge is four inter-urban tracks, six lanes of roadway, and two footways.

It is considered that the bridge is now used to capacity and there is an increasing need for further cross-harbour facilities.

2. *Auckland Harbour bridge.* This cannot be considered a long-span bridge, but it is of interest as another major bridge in the Commonwealth now being built as a self-liquidating project. It is a highway bridge (Fig. 9) with an overall length of 3,348 ft across Auckland Harbour, and has as its object the development of the north-shore suburbs and provision of direct access from the north shore to the city.

The Royal Commission appointed to report on trans-harbour facilities in 1946 recommended the construction of a bridge with a four-lane carriageway. It estimated that the traffic on the bridge would be as follows:—

1956 . . .	5,500	vehicles per day, i.e.,	1,950,000 per annum
1965 . . .	8,250	,, ,, ,, ,,	2,900,000 ,, ,,
Ultimately .	26,800	,, ,, ,, ,,	9,200,000 ,, ,,

Authorities in the U.S.A. state that the practical or working capacity of a four-lane highway may be rated at 28,000 vehicles per day. The Commission found that on the basis of the cost of the bridge and approaches not exceeding £3,000,000, and assuming a scale of toll charges (e.g., 1*s* per five-seater car) fixed at about half the rates charged by the ferries, the bridge could be built as a self-liquidating project.

Because of post-war political difficulties in arranging the finance, the contract was not finally concluded until 1954 and the overall cost had then risen to about £5,000,000. In spite of this there is still no difficulty in making the bridge a paying proposition; but without the imposition of tolls it could not have been afforded for many years, and indeed might never have materialized.

Examples of toll bridges and tunnels in the U.S.A.

1. *Golden Gate bridge (1933–37). Suspension span 4,200 ft.* This bridge, which is the longest single-span yet built, was financed as a self-liquidating project by means of a $35,000,000 bond issue; the bonds are non-callable and finally mature in 1971. The bridge has been a financial success from the start and has paid its own way from the opening date.

The annual revenue, less operating and maintenance expenses, is of the order of $3,300,000, and it is estimated that on this basis the bridge could be freed if desired within 13 years.

The toll charges have been adjusted downwards during the years and at the present time a private car pays a one-way toll of 30 cents.

The number of vehicles using the bridge has increased from 3,300,000 in 1938, the first full year of operation, to 12,300,000 in 1954. The North Bay area served directly by the bridge has maintained steady growth and development.

The bridge carries six lanes of roadway traffic and two footways.

2. *San Francisco–Oakland Bay bridge (1933–36).* The Bay bridge comprises two

suspension spans of 2,310 ft each, a cantilever bridge of 1,400-ft span, and a tunnel in Yerba Buena Island. The overall length is about 4½ miles and the total cost $79,500,000. It is one of three bridges owned and operated by the State of California, all of which were financed by revenue bonds secured by bridge revenue only. The revenues have always substantially exceeded the estimates and would in fact have sufficed to pay all maintenance, operation, and interest charges and also for the initial cost of the bridge by 1954 if they had been allocated to this purpose.

However, present Californian law provides that with the redemption of all outstanding Bay-bridge bonds, the State Highway Fund is to be reimbursed out of bridge revenues for all past expenditures which it shall have incurred for Bay-bridge operation and maintenance, and that thereafter tolls shall continue to be collected to cover such continuing costs. Studies for a second crossing of the Bay are now being made, their cost being paid from Bay-bridge revenue. If the second crossing is built, tolls will be continued on the present bridge to assist in paying for the new structure.

Successive reductions have been made in toll charges for private cars as follows:—

November 1936 . . .	Started at	65	cents
February 1937	Reduced to	50	,,
June 1939	,, ,,	40	,,
January 1940	,, ,,	35	,,
May 1940	,, ,,	30	,,
July 1940	,, ,,	25	,,

The volume of traffic has increased from 9,100,000 vehicles in the first year of operation to 31,200,000 in 1954. Although it has been in use less than 20 years, the bridge is now used to capacity and steps are being taken to provide an additional crossing.

The Bay bridge is double-decked and carries six lanes of roadway traffic on the upper deck and originally carried three lanes for trucks and buses on the lower deck. In 1939, two additional tracks for inter-urban railways were opened on the lower deck.

3. *George Washington bridge (1931)*. Built at a cost of $75,000,000, this bridge, which has a suspension span of 3,500 ft, is one of four constructed and operated by the Port of New York Authority. Besides these four bridges, the Port Authority facilities include two tunnels, seven marine and inland terminals, and four airports. Its financial structure is such that the revenues of each facility are combined and used to pay the principal and interest on a consolidated bond issue. It is difficult therefore to determine the financial position of any individual facility, but the successful handling of an ever-increasing amount of traffic and the plans made jointly by the Port Authority and the Triborough Bridge and Tunnel Authority to build and operate additional crossings in the New York Metropolitan area indicate a dynamic and prosperous condition.

The traffic volume in 1953 was 30,800,000 vehicles, which exceeds the estimated capacity of the upper deck. The charge for a private car is 50 cents.

At first only sufficient of the roadway on the upper deck was constructed to carry four lanes of traffic, but some years ago the full width was completed and it is now capable of carrying eight traffic lanes.

The Authority's recommended programme provides for the construction, by 1960, of the six-lane lower deck of the bridge at a cost of $82,000,000.

4. *The Triborough bridge* (*1936*). This bridge is a combination of viaducts and spans connecting the New York boroughs of Queens, the Bronx, and Manhattan. It comprises $3\frac{1}{2}$ miles of bridge and 14 miles of approaches, the total cost being \$63,000,000. It is one of five bridges, two tunnels, and numerous other facilities constructed, maintained, and operated by the Triborough Bridge and Tunnel Authority. It is estimated that the present debt of the Authority, covering all its facilities and totalling \$215,000,000, could be extinguished in 1969, i.e., in 13 years' time.

The volume of traffic on the Triborough bridge has increased from 11,000,000 vehicles in 1937, the first full year of operation, to 38,100,000 in 1954, and is approaching the capacity of the bridge. The charge for private cars is 25 cents.

The bridge carries eight lanes of traffic from the Bronx to Queens and six lanes between Randall's Island and Manhattan.

5. *Bronx–Whitestone bridge* (*1939*). *Suspension span 2,300 ft.* The total cost of this bridge, which is one of the group constructed and run by the Triborough Bridge and Tunnel Authority, was \$18,000,000 and it is estimated that it could, if desired, be freed within 14 years.

The volume of traffic has increased from 6,300,000 vehicles in 1940 to 26,000,000 in 1954 and is nearing the capacity of the bridge. The charge for a private car is 25 cents.

When first opened the bridge carried four lanes of roadway traffic but was widened to carry six lanes in 1946.

6. *Tacoma Narrows bridge.* The first bridge was opened in July and collapsed in November 1940, after only 4 months' use. Its cost was \$6,400,000. The second bridge, which had a suspension span of 2,800 ft, was opened in 1950 at a cost of \$14,000,000 and incorporated material worth about \$3,500,000 from the old bridge. Both bridges were financed by means of revenue bonds issued by the Washington Toll Bridge Authority. Tolls will be charged on the bridge long enough to repay all moneys expended on interest, operation, and maintenance, and to redeem the bonds.

The charge per car and driver was fixed on both the old and new bridges at 55 cents plus 15 cents per passenger. In 1953, it was reduced to 50 cents plus 10 cents per passenger. Daily users can get reduced fares amounting to about 60% of the above.

Making allowance for the "curiosity traffic" during the first month of operation, it was estimated that if the first bridge had continued to operate it would have carried in the first year 2·3 times the amount of traffic the ferry system would have carried had it remained in service. In other words, the traffic induced by reason of having a bridge instead of a ferry was 130%.

After the bridge was destroyed, the ferry line was re-established and operated by the Washington State Highway Department. That the bridge had already had a major influence on the traffic pattern of the community was demonstrated by the fact that the annual traffic rate on the ferries for the year in which the bridge was opened totalled 245,000 vehicles, and for the year after the bridge was destroyed it amounted to 460,000. The State continued to operate the ferries until the new bridge was opened to traffic. They provided for better and more frequent service than was given by the original operators. During the war years the traffic increased very little, but in the first full post-war year the ferries carried 679,000 vehicles, or 20% more than the yearly traffic rate established during the period of bridge operation. This growth was due in part to the population increase and to the increased

post-war rate of automobile use. In addition there was continuous development in the areas west of Tacoma Narrows. In spite of all of this abnormal traffic growth on the ferries, the second bridge when opened drew approximately 100% more traffic than the ferries had been handling.

Both bridges carried four lanes of roadway traffic.

7. *The Holland tunnel.* Before 1927, when the Holland tunnel was opened to traffic, ferry boats provided the only means for vehicles to cross the Hudson River between the New Jersey and New York portions of the metropolitan area. The Holland tunnel, which cost about $50,000,000 to construct, runs from lower Manhattan to Jersey City. It is, in fact, two tunnels providing two traffic lanes in each direction. Each tunnel is over 9,000 ft long, with carriageways 20 ft wide.

The following figures show the total number of vehicles using the tunnel annually:—

1941	14,231,415
1946	15,351,332
1947	15,463,192
1948	15,600,124
1949	16,484,014
1950	18,125,787
1951	19,633,947
1952	18,782,343
1953	19,443,674

The total income from tolls in the year 1950 was $10,489,863, which is about 58 cents per vehicle.

8. *Lincoln tunnel.* The Lincoln tunnel, also under the Hudson River, connects mid-town Manhattan with the New Jersey section of the metropolitan area. This also consists of two separate tunnels with two lanes in each direction, the roadways being 21 ft 6 in. wide. The first tunnel was brought into use for two-way traffic in December 1937. Construction of the second tunnel was started in 1937 but suspended in 1938, because the through highways in New Jersey had not been constructed. Following an increase in traffic, work was resumed in 1941 and the tunnel was opened in February 1945. Each tunnel then became a two-lane thoroughfare. The two tunnels cost about $65,000,000.

The total number of vehicles using the tunnel each year was:—

1941	4,681,157	(only 1 tunnel in operation)
1946	9,610,114	
1947	10,635,383	
1948	11,121,107	
1949	12,962,842	
1950	15,532,561	
1951	17,462,312	
1952	19,577,039	
1953	20,771,766	

The average toll paid by each vehicle was about 59 cents in the year 1950, when the total income was $9,223,621.

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Part 3

TURNPIKES

by

E. W. W. Richards, A.M.I.C.E.

Introduction

Turnpike roads have developed from the simple fact that the tremendous growth in the use of motor transportation has outstripped both the capacity of the ordinary highways to accommodate it and the ability of governments to provide the funds for necessary improvements and new construction. In such circumstances, it has proved possible, in the United States and many other countries, to finance the construction of such highways through the sale of bonds serviced solely from the tolls received. By collecting from the user of the highway the cost of maintaining and operating it and of servicing and paying off the debt, it has been possible to construct many thousands of miles of limited-access highways of the highest standard.

A turnpike highway system stands in a complementary relation to the non-toll-paying highway system of its area. On the one hand, it is a competitor. On the other hand, it relieves the non-toll-paying highway system of sufficient traffic to make its use more convenient. The non-toll-paying highway system serves as a feeder for the toll turnpike which, with its limited access, is of necessity a trunk system and not a distributor. The turnpike road serves, essentially, passenger and commercial traffic which can obtain the benefit of high-speed trips of reasonable length, averaging, say, 30 miles. Trips under 4 or 5 miles do not usually justify the travel necessary to reach a turnpike entrance from a particular origin and to reach a particular destination from a turnpike exit. Experience on the New Jersey turnpike—the most heavily trafficked turnpike in the United States, stretching 118 miles between the southern end of New Jersey and New York—has shown that over the entire length only 5% of vehicles travel the full distance of the turnpike from end to end. Indeed, it would seem that the advantages of the turnpike may be estimated in terms of time saved rather than distance travelled, and that, in the three access points within the New York section of the New Jersey turnpike, no less than [illegible]0% of all this traffic leaves the turnpike again within this limited, but heavily trafficked, turnpike area.

The rates of toll on the various turnpikes constructed in the United States vary from 1 to 3 cents a mile for passenger cars, but it is general experience that, given a

choice of the use of a free road or a turnpike, about three-quarters of the potential users of either are willing to pay $2\frac{1}{2}$ cents (roughly $2\frac{1}{2}d$) for each minute that can be saved by the use of a toll road. To commercial traffic there are added attractions in the elimination of distance, or the improvement of gradients, and commensurately higher toll charges, ranging up to three or four times passenger-vehicle rates, still leave sufficient inducement to attract large volumes of commercial vehicles. It would, however, be a mistake to assume that turnpike roads principally attract, or are intended usually to serve, commercial or load-carrying traffic. It is a fundamental assumption, in the American view, that it is human beings carried in passenger vehicles who conduct business and create prosperity, and that they require protection from the strain and hazard of driving on overcrowded and unsatisfactory roads, as much as any reduction in the cost of transporting goods.

Toll roads and bridges have existed in the United States for a long time, and do not appear to have aroused the animosity which they have been known to excite in our own country. The advantages offered include relief from danger and delay through congestion; relief from delay and extra expense through uneconomic alignment; a reduction in the accident rate; improved amenities through careful layout and planning; and almost complete freedom from the risk, strain, and anxiety of normal travel.

Organization of the turnpike system in the U.S.A.

The modern turnpike road system originated and was developed principally in the United States and, though its organization and structure have been copied, with local variations, in many countries, it is still the prime exemplar, and an account of its administrative and constructional features, many of the former originating from British Common Law, must serve as a basis for any consideration of the subject.

The Turnpike Authority, in the United States, is created by the State and is authorized to construct specific types of highways and to issue securities to cover the cost of construction and operation of the road. Broadly, it has, among its other powers, the ability to require land by negotiation or compulsory purchase, though it has no power to commit the State as to the payment of interest or principal on any of its debt. The Authority is constituted of a group of from three to seven men, usually eminent industrialists, operating under the direction of a chairman and possessing the requisite staff to administer its functions. The law creates the Authority, and defines its duties and powers, which include permission to construct, maintain, repair, police, and operate a toll-road project, or projects within the State; to issue bonds in the name of the Authority for that purpose and to retire those bonds, together with interest, solely from toll and other revenues. The Authority is empowered to employ engineers and such other experts as may be necessary and to fix their remuneration.

Estimates of traffic and revenues

The Authority then assigns engineers to make surveys and reports, preliminary to the completion of the financing. The first assignment has to do with the surveys required for the preparation of estimates of traffic or use of the proposed facilities, the determination of suitable charges or tolls for their use, and estimates of revenues. The importance of this report is obvious from the fact that the bond owner must rely solely on the net revenues for all interest payments and debt retirement. Of equal importance are the preliminary location, plans, and cost estimates.

Comprehensive origin-destination surveys are conducted at carefully selected

stations on thoroughfares carrying traffic which might use the new facility. These surveys are generally conducted by the roadside-interview method and cover both passenger and commercial vehicles. At the same time, volume, or classification counts are made and information as to running time is collected.

The next step is the preparation of estimates of the percentage of traffic between each pair of zones in the area which would be diverted from the existing highway to the new toll road. The principal factors in arriving at these estimates are time savings, reductions in length of travel, improved safety, and increased convenience. These estimates must be based on the records of other similar projects reviewed in the light of past experience.

This economic study and report will also contain recommendations for an initial schedule of tolls, and these are founded on analyses of the likely results of application of varying toll schedules. Tolls on turnpikes, for instance, are generally considered at rates varying from approximately 1 cent to 3 cents per mile for passenger vehicles. Initially, in predicting the traffic and revenues in the first year of operation of a toll facility, the diversions from the free highway system in the area to the new toll facility are estimated. Present volumes on such routes are utilized to determine total traffic and revenues during the initial year.

These figures of first operation are prepared without any consideration of the new and additional traffic which experience shows is invariably generated by improved facilities. The amount to be added for generated traffic is generally substantial on turnpikes, ranging from 10 to as much as 40% over a period of 5 or 6 years from the start of the operation. There is no precise way for arriving at these figures. They are generally determined on the basis of judgement after consideration of the general characteristics of the area. The New Jersey turnpike, opened for traffic in November 1951, was a spectacular example of the volume of traffic which may be induced with the provision of a first-class facility in a district where the traffic pressure is very great.

In addition to toll revenue, there is also the revenue derived from the operation of various facilities appurtenant to the operation of the project. In the case of a urnpike, these consist of filling stations, snack bars, and restaurants located on the and belonging to the public agency and either built by the agency itself or by a oncessionaire. The return to the toll authority is generally based on a contract providing for payment of a percentage of the gross business of the concessionaire. The return from concessionaires on turnpikes is substantial, amounting to some 5% of the revenues from tolls. To establish a sound basis for revenue-bond financing, t is necessary to make projections of traffic and revenues for a considerable period in he future. As a rule, projections are extended throughout the life of the proposed ond issue, which may be as much as 40 years. The factors utilized in these fore-asts include population growth, increases in motor-vehicle ownership, increases in otor-vehicle use, development of competing facilities, and finally, trends in in-ustrial, agricultural, and mining activities in the district and surrounding region.

Accurate records are generally available to establish past trends and these con-titute a reasonably satisfactory base for projecting traffic forecasts. The annual icreases have been and will continue to be substantial. This is important from the iewpoint of financing. It is frequently found that the net revenues in the initial ear of operation are barely sufficient to pay interest charges, but that with the icreased revenues after the first few years, and particularly in the later years of ie period covered by the bond issue, revenues will provide amply for all interest id amortization charges. In the majority of instances, amortization of revenue

bond issues for toll roads, bridges, and tunnels has been completed in advance of schedule and in some cases the growth in revenue has been phenomenal.

Layout and design

Estimates of future traffic also have an important and useful function in providing the base for the constructional and geometric design of the project. The designing engineer must be guided by the estimates of future peak traffic in the determination of such important items on turnpikes as, for example, the width of the roadways, the number and location of interchanges, and the layout of toll plazas. It is sometimes found that a four-lane highway will be adequate for all traffic anticipated during the first few years of operation, for instance, but that the increases in traffic during the life of the bonds will be such as to require two additional lanes in later years In such cases it is customary to design a wide centre median of, say, 50 to 60 ft so as to permit the subsequent construction of two 12-ft lanes adjacent to the interior edges of the pavements. Such additional strips can be built without any major interruption of traffic on the turnpike and without any disturbance to the acceleration and deceleration lanes or to the roadways leading to the interchanges or ramps at the sides of the turnpike.

The engineering report is the basis for the determination of the amount of the bond issue. The report is founded on a preliminary examination, location, and design. The preliminary examination includes soils, geology, drainage, topography, and transportation and access for constructional personnel, equipment, materials, and supplies. Studies are also made of intersecting and parallel utilities, cross-roads, railroads, land use, and connecting facilities at proposed terminals. The preliminary location is made and both line and grade are established. The availability and cost of materials and equipment to be used in the construction are determined and complete design criteria are prepared. Finally, preliminary designs of the entire facility, including interchanges, terminal connexions, carriageways, toll-collection facilities, maintenance and headquarters buildings, yards and equipment, and concession areas are developed. These designs are utilized as a basis for estimates of construction costs.

An important part of the engineering investigation is the determination of the width of right-of-way, including rights-of-way and easements required for the construction of cross-roads, utilities, and railroads in the case of turnpikes. Other items to be covered in the engineer's estimate include maintenance shop, police, and office equipment, and sometimes concessions.

Financing the project

It is also necessary, at this stage, to make a preliminary estimation of all project costs, including preliminary costs, land acquisition, construction, utility adjustments, engineering, legal, and administrative costs, maintenance equipment, contingencies, interest during construction, and financing cost. The sum total of all of these estimates determines the amount of the revenue-bond issue. During this period, in addition to the economic and engineering studies, a great deal of legal and financial work must be accomplished. The lawyers retained by the toll road authority must test the constitutional soundness of the turnpike law as well as prepare to resist any litigation. With the Authority's financial advisers, the legal officers prepare the trust indenture which secures the bonds, as well as the bond prospectus or official statement. The investment bankers set up the amortization schedule and the flow of funds to the several reserve accounts.

Fig. 10.—Section of New Jersey turnpike
Final surfaces consist of three layers of hot-mix asphaltic concrete

Fig. 11.—A three-level crossover under construction

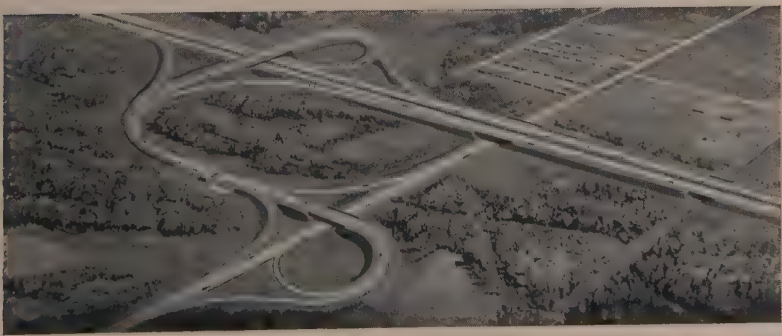

Fig. 12.—A typical interchange at junction with New Jersey turnpike

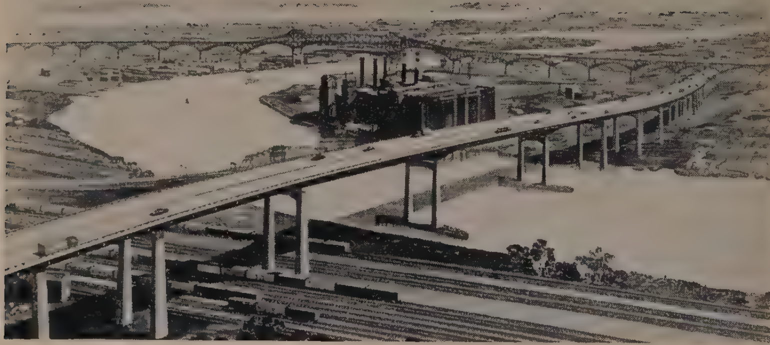

Fig. 13.—A 7,000-ft six-lane viaduct (passing under the Pulaski Skyway)

Fig. 14.—Typical toll-gate at the entrance to the turnpike

Fig. 15.—A typical lunch-room

FIG. 16.—TRUCKS AT TOLL PLAZA ON OAKLAND BAY BRIDGE, SAN FRANCISCO

FIG. 17.—AERIAL VIEW OF TOLL PLAZA AT OAKLAND BAY BRIDGE. THE SIXTEEN LANES AT WHICH TOLLS ARE COLLECTED ARE TO BE INCREASED TO THIRTY.

When all this has been done, the engineering reports and the official statements are distributed to all interested parties and information meetings are arranged, usually in New York and Chicago, at which the project may be explained to those concerned. To these information meetings come representatives of the country's large insurance companies and from the investment-trust field, who ask searching questions regarding the project and the all-important matter of "coverage." "Coverage" is the ratio between estimated annual earnings and annual debt-service requirements. If this ratio averages 1½ to 1 over the life of the bond issue, the bonds will certainly be sold. If, on the other hand, the estimated net revenue does not exceed the debt service by at least 50%, the project will undoubtedly be tabled awaiting the time when traffic will be sufficient to produce the necessary annual income. Within the past year engineering studies indicated that toll-road projects should be tabled in the States of Wisconsin, Ohio, and Oklahoma.

Once a toll-road project is successfully financed, the Authority retains its Chief Engineer and the proposed toll-road route is divided into sections for contract plans and specifications as well as supervision of construction. These sections generally are in the neighbourhood of 20 miles in length. Contracts are then given to as many engineering firms as there are sections, so that all work proceeds simultaneously, thereby accomplishing great savings in time, which is of paramount importance since the bonds have been sold and are earning interest daily. In the case of the Ohio turnpike the daily interest amounted to about $30,000 (£11,000). Time is here truly the essence of the contract.

The interest of the financial and investment agencies in the turnpike system is shown by the fact that since 1946 (and since the construction of the Pennsylvania and Maine turnpikes) a total of $3,200,000,000 (£1,170,000,000) turnpike bonds has been issued to finance nearly 3,000 miles of road. The growth of the turnpike system and the confidence of investors in the self-justification of highway work, may be better seen from the fact that (in addition to Federal and State expenditure at the rate of about $6,500,000,000 (£2,575,000,000) *a year* on public highways) the total sum spent or committed in the American turnpike system (January 1955) for projects built, under construction, and authorized was no less than $8,700,000,000 (£3,190,000,000) for the construction of 8,500 miles of controlled-access superhighways.

It will be interesting to examine in some detail two modern American toll roads, built for two different reasons; one for relief from the consequences of uneconomic alignment and for the development of trade, and the second for relief from traffic congestion (the Pennsylvania and New Jersey turnpikes respectively) to see whether they have produced these benefits, and whether public recognition and response have been satisfactory.

The Pennsylvania turnpike

The Pennsylvania turnpike system was originally constructed to breach the natural geographic barrier of the Appalachian Mountain ranges which lie between the ports and the highly developed coastal areas of the American Atlantic seaboard and the increasing industrial and agricultural potential of the Middle West. This has been achieved by piercing the highest ridges of the Appalachians with seven major tunnels of a combined length of 35,000 ft, by spanning 274 streams and rivers (the longest bridge being that over the Susquehanna at Harrisburg, which is 4,600 ft in length) and by the construction of 378 structures for carrying non-intersecting

highways over or under the turnpike and the provision of twenty-four major interchanges at points where access to the turnpike is permitted. To facilitate the movement of vehicles upon this unrestricted highway, its alignment and curvature were so designed that no gradient should exceed 1: 33, and no curve should be of less radius than 1,000 ft. The minimum visibility distance, whether arising from curvature or gradient, has been so regulated that it is possible to see a brick on the road a quarter of a mile away. The width of the turnpike, varied over bridges and through tunnels, is 78 ft in open country and comprises two one-way carriageways each 24 ft in width, separated by an intervening median strip of 10 ft, and bordered by hard shoulders on which a vehicle may either pull off to park or on to which it may run if it gets out of control. It is forbidden to park, stop, or load on traffic lanes, deceleration or acceleration lanes, at bridges, structures, or on the road in front of service stations, which are located off the highway and provided with adequate car parks. U-turns are not allowed, nor are pedestrians or cyclists permitted. There is, therefore, an unimpeded and unrestricted flow of high-speed traffic along the whole route. At one time unrestricted speeds were permitted, but as some casualties were caused (to the drivers) by vehicles getting out of control at excessively high speeds, a speed limit has been imposed for passenger cars, motor-cycles, and buses; the speed limit for commercial lorries is slightly lower. Throughout the turnpike the motorist is served by attractively located and designed repair and service stations and restaurants operated under concessions. In addition to an elaborate system of patrol and repair vehicles, the turnpike is completely covered by radio communication and, where appropriate, with teleprinter services.

In 1948, comparative tests for petrol consumption and for speed were conducted on the Pennsylvania turnpike between Carlisle and New Stanton interchange, a distance of 148·7 miles, and on a parallel route consisting of U.S.11 from Carlisle to Chambersburg and U.S.30 from Chambersburg to Greenburg, near New Stanton interchange, a distance of 149·4 miles. Although covering approximately the same distance as the turnpike, the free road had much variety in surface type, width of pavement, gradient, and curvature. Tests were made both eastbound and westbound with seven different classes of trucks and three different gross weights in each class. The speed limit was the legal one in each case, 50 m.p.h. on the turnpike and 30 m.p.h. on the free road. A representative selection of eleven vehicles from this study, ranging from 20,030 lb. to 139,500 lb. in gross weight, revealed that the average petrol consumption was 3·77 m.p.g. on the turnpike and 2·35 m.p.g. on the free road, and that average speed was 38·9 m.p.h. on the turnpike and 21·2 m.p.h. on the free road. The average petrol consumption on the free road was 61% greater per mile than on the turnpike and the average speed per hour was 83% higher on the turnpike.

The savings in time and petrol are obvious to the driver, but some of the other economies are not so immediately apparent. Garage mechanics along the Pennsylvania turnpike report less wear and tear on vehicles than on parallel roads, brakes are used less, fewer tires blow out owing to easy curves, vehicle engines do not overheat, and oil consumption is low.

Construction of the highway began late in 1938, and the first section, 160 miles in length, was opened to traffic in October 1940. With subsequent extensions the total length of the turnpike is now 327 miles.

During the first 10 years after the original 160-mile section of the turnpike was completed, it has been recorded that almost 2,500,000,000 miles were covered by 25,000,000 motorists on the turnpike system, yielding an income of $42,000,000

:15,100,000) in fares; an average of 100 miles a journey at $1\frac{1}{2}$ cents (pence) a mile. n analysis of travel during this period shows that passenger cars totalled ,736,890,876 miles at an average of slightly less than 1 cent (penny) a mile, com- iercial trucks accounted for 610,803,258 miles at an average of a little over 4 cents mile, while bus operators travelled 25,915,766 miles at an average of $3\frac{1}{2}$ cents a iile. This includes the war period, during which the turnpike served as a major iilitary highway on which private travel was restricted. Since the ratio between assenger travel and goods transportation throughout the United States is approx- nately 6:1, it would appear that travel on the turnpike is about twice as attractive ɔ industrial transport as to the private user, in spite of proportionately higher fares. n 1950, during which all previous annual records were broken, 4,488,548 fare-paying iotorists travelled 453,277,352 miles, a rate nearly double the previous 10-year verage, and yielded a revenue of $9,022,398 (£3,200,000). The month of March 951 surpassed all previous records. A total of 497,545 vehicles used the turnpike, s compared with 224,249 for the same month a year earlier. During this month, evenues totalled $1,010,765 (£360,000), representing a gain of 73% over the same ionth in the previous year. Obviously, the advantages of the turnpike were roving increasingly attractive to highway users in a period of unrestricted travel.

With the exception of an initial and comparatively small grant from the Re- onstruction Finance Corporation towards the construction of the original turnpike, he cost of the whole system has been found from privately subscribed funds. Not ne penny has been found by the taxpayers, though the turnpike will eventually evert to them as a free road. The road has cost, and has been financed as follows:—

Original section:		
Carlisle to Irwin (160 miles)	$76,250,000	£27,500,000
Eastern extension:		
Carlisle to Valley Forge (100 miles) . . .	87,000,000	31,300,000
Western extension:		
Estimated cost Irwin to Ohio Border (67 miles)	77,500,000	28,000,000
	$240,750,000	£86,800,000
Grant from Public Works Administration for 160-mile original section	29,250,000	10,500,000
Bonds held by investors	$211,500,000	£76,300,000

Without involving the faith and credit of Federal or State Government, the riginal bonds, due for retirement at periods between 1952 and 1968, bore interest at $\frac{3}{4}$% and a subsequent issue, to finance extensions, was made in 1949 at $3\frac{1}{4}$%. In ctober 1950, all these turnpike bonds were selling at substantial premiums over ıeir original offering prices. Statistical information on the use of the Pennsylvania urnpike, the traffic it carries, and the cost of operating it are contained in Tables 6, , 8, and 9.

Table 6 shows the relative importance, annually, of the different classes of vehicle sing the turnpike and the proportion of the total toll revenues contributed by each lass. Ignoring the early years and the war years, passenger cars averaged almost our-fifths and trucks roughly one-fifth of the total number of vehicles using the toll

road. With regard to toll revenue, during and since the second world war, trucks have contributed more than half of the total amount.

Table 7 shows the different sources of revenue. Tolls account for almost the whole amount and the proportion is fairly uniform from year to year. The heading "Concessions" refers to rent from leases and concessions granted to the twenty-two service stations and restaurants along the turnpike.

Table 8 gives the distribution of the total operating expenditure. The proportion of expenditure on tunnel maintenance has declined appreciably of recent years due

TABLE 6.—DISTRIBUTION OF TRAFFIC AND TOLL REVENUES ON THE PENNSYLVANIA TURNPIKE ACCORDING TO CLASS OF VEHICLE

Year	Total: × 1,000	Vehicles: % of total			Total (thousands of dollars)	Toll revenues: % of total		
		Passenger cars	Trucks	Buses		Passenger cars	Trucks	Buses
1941	1,274	87·5	11·8	0·7	1,418	60·2	38·4	1·4
1942	2,354	86·2	12·7	1·1	2,875	57·4	40·1	2·5
1943	1,120	70·1	28·7	1·2	1,879	28·0	69·8	2·2
1944	954	67·1	31·9	1·0	1,708	25·7	72·3	2·0
1945	1,051	68·4	30·3	1·3	1,819	26·8	71·1	2·1
1946	1,891	80·6	18·1	1·3	2,689	46·1	50·9	3·0
1947	2,577	80·1	18·7	1·2	3,801	44·8	52·8	2·4
1948	3,143	79·1	19·9	1·0	4,844	42·0	55·9	2·1
1949	3,597	77·3	21·5	1·2	5,957	38·5	59·5	2·0
1950	3,962	76·0	23·1	0·9	7,173	35·6	62·9	1·5
1951	5,495	76·1	23·2	0·7	10,127	37·0	61·7	1·3
1952	8,792	81·0	18·4	0·6	15,162	44·4	54·4	1·2
1953	11,304	80·5	18·9	0·6	19,976	43·3	55·6	1·1

TABLE 7.—RELATIVE IMPORTANCE OF THE DIFFERENT SOURCES OF OPERATING REVENUES

Year	Total operating revenue: × 1,000 dollars	% of total		
		Toll revenues	Concessions	Miscellaneous revenues
1941	1,529	92·7	7·3	(included with concessions)
1942	3,091	93·0	7·0	(,, ,, ,,)
1943	1,969	95·1	4·2	0·7
1944	1,785	95·6	4·0	0·4
1945	1,906	95·4	4·2	0·4
1946	2,905	92·6	7·2	0·2
1947	4,120	92·3	7·4	0·3
1948	5,187	93·4	6·3	0·3
1949	6,325	94·2	5·7	0·1
1950	7,550	95·0	4·9	0·1
1951	10,690	94·7	5·2	0·1
1952	16,185	93·7	6·2	0·1
1953	21,261	93·9	6·0	0·1

o the fact that all seven tunnels are on the original sector. With the opening of the Philadelphia and Western extensions, the expenditure on tunnels became thus a portion of a much larger total expenditure.

Table 9 presents a summary of income and its distribution to bond interest and replacement reserve fund. Appropriations to the latter are for the purpose of establishing a fund that may be drawn upon to replace the turnpike and its equipment.

TABLE 8.—RELATIVE IMPORTANCE OF THE DIFFERENT CLASSES OF OPERATING EXPENSES

Year	Total operating expenses: × 1,000 dollars	% of total					
		Maintenance roadway and structures	Maintenance and operation of tunnels	Toll collections	Turnpike patrol	Stores and suspense cost items	General and administrative
1941	651	*	*	*	*	*	*
1942	839	*	*	*	*	*	*
1943	814	35·7	20·9	13·7	8·9	—	20·8
1944	781	35·7	21·4	13·8	8·4	0·5	20·0
1945	745	38·9	19·3	15·4	5·4	0·5	20·5
1946	779	38·3	19·0	16·8	5·6	1·2	19·1
1947	999	39·0	17·5	17·1	7·9	1·1	17·4
1948	1,186	37·1	20·4	16·4	7·2	0·8	18·1
1949	1,334	36·5	19·1	16·8	7·4	1·1	19·1
1950	1,315	33·5	20·4	18·3	8·0	0·9	18·9
1951	1,913	39·1	15·5	20·9	7·3	1·0	16·2
1952	3,311	42·5	9·5	23·0	7·2	0·7	17·1
1953	4,364	41·5	7·3	17·5	5·4	0·5	19·8

* Data not available.

TABLE 9.—SUMMARY OF INCOME AND OTHER DEDUCTIONS

Year	Total net income after deducting all expenses: × 1,000 dollars	Bond interest	Replacement reserve fund
1941	878	—	—
1942	2,252	1,020	—
1943	1,160	1,530	12
1944	1,009	1,554	5
1945	1,164	1,586	9
1946	2,126	1,586	—
1947	3,126	1,250	2
1948	4,031	1,144	46
1949	4,996	834	218
1950	6,240	1,058	187
1951	8,781	2,236	157
1952	12,879	4,778	1,257
1953	16,905	5,891	3,383

The New Jersey turnpike

The New Jersey turnpike (see Figs 10, 11, 12, 13, 14, and 15) is of more recent conception and construction than its Pennsylvania counterpart and was built to deal with a different type of problem. The State of New Jersey, though only forty-fifth in size in the United States, ranks sixth as an industrial producer and second in density of population. It forms part of a group of north-eastern states comprising 35% of all American industry and contains part of the New York–Philadelphia metropolitan areas in which the population numbers about 14,000,000 or about 10% of the entire population of the country. The State is elongated in shape, lying parallel to the Atlantic seaboard, and practically all highway traffic to and between the great seaports of New York and Philadelphia passes through it. These factors have created a transportation problem almost without parallel in the United States.

The highway problem in New Jersey became acute shortly after the first world war, when an ambitious programme of highway construction was initiated which included the construction of the famous Pulaski Skyway, major approach highways to the entrances to Philadelphia and New York, and the construction of U.S. Route No. 1 from the Holland Tunnel, New York to Trenton, New Jersey. These projects embraced a substantial portion of the total dual and dual-dual highway milage in the United States. By the early 1930s the State had almost caught up with traffic requirements as far as capacity was concerned. When the depression set in, highway construction was curtailed. This curtailment lasted through the second world war, piling up a 20-year backlog of highway improvements. In the meantime, traffic had continued to increase, and since 1940 has gained 40 to 50% in volume. The result was that the State's highways were required to carry more traffic than they were designed for—in some cases more than double the capacity that might be considered reasonable—giving rise to extreme congestion and delay.

In 1946, the State Highway Department made a comprehensive survey of minimum highway needs to provide for the existing traffic level and found that an expenditure of $600,000,000 (£215,000,000) would be required, but nothing was done. Early in 1948, upon urgent private representations, the State Governor presented to the State Legislature a proposal to create a Turnpike Authority to finance and build trunk roads by the sale of revenue bonds to private investors. The Act was passed and became law in October 1948, and by it the New Jersey Turnpike Authority was established as an instrument of the State exercising essential public and governmental functions "to construct, maintain, repair and operate turnpike projects at such locations as shall be established by law, and to issue turnpike revenue bonds of the Authority, payable solely from tolls, other revenues, and proceeds of such bonds to finance such projects." Such bonds "shall not be deemed to constitute a debt or liability of the State or of any political sub-division thereof or a pledge of the faith and credit of the State or of any such political sub-division."

On the 31st March, 1949, the State Governor appointed three outstanding businessmen as Commissioners to the Authority, offices were set up, and staff was recruited. The State Highway Engineer was seconded to the Authority as Chief Engineer. In May 1949, the Commissioners ordered four engineering firms to reconnoitre this complicated route, extending for 118 miles from the borders of New York to the Delaware River through the most highly developed and industrialized territory in America, to prepare engineering, traffic, and revenue studies of an initial proposal within a time-limit of 120 days. This schedule was met. The preliminary conclusions of the engineers indicated that the turnpike would be financed as a self-

supporting project and that its cost would be about \$230,000,000 (£82,500,000). *Within a week it had been decided to proceed with the project, and instructions had been given for the preparation of contract plans and specifications, land acquisition and the supervision of construction. The time allowed for completion of the entire project, including these preliminaries, was 2 years. This schedule was also met.*

In financing the project, which was heavily subscribed by the main American insurance companies, the preliminary engineering reports had indicated that an overall sum of \$230,000,000 (£82,500,000) would be required. This cost later grew to approximately \$280,000,000 (£100,000,000) mainly through increases in costs of materials. The plan adopted was on a "forward commitment" basis. It provided a call on the full \$220,000,000 for which the Authority paid a nominal fee of $\frac{1}{2}$% annually. As funds were required, a draft was made against the "commitment" and definitive bonds bearing interest of $3\frac{1}{4}$% annually were issued for the amount required. The bonds mature in 35 years, in accordance with the terms of the act, but traffic surveys and previous experience of toll road and bridge operation indicate that it will be paid off in less than that period. The law then requires that the highway be turned over to the State Highway Department, though not one penny of taxpayers' money will have been used in financing its construction and operation.

It is estimated that with a traffic density of 7,500,000 vehicles using the turnpike in the first year, rising to over 10,000,000 a year within 5 years, and increasing steadily thereafter, the bond issue will, in fact, have amortized itself in 24 years, though the period of issue is for 35 years, by which time the turnpike will have paid for itself 1·62 times over. These estimates of traffic volumes have been greatly exceeded.

The engineering features of the New Jersey turnpike are very similar to those in Pennsylvania, though provision is made in the vicinity of the large towns for carrying very large volumes of traffic on dual triple-lane separated carriageways and even for dual-dual or 8-lane divided operation. The turnpike is a controlled-access highway throughout, with 164 grade separation crossings and seventeen interchanges, including those of the terminals. Acceleration and deceleration lanes at interchanges and service areas are 1,200 ft long to allow merging traffic to adjust its speed to that of the traffic on the lane it will be joining. Service areas comprising motor-repair and service stations, lunch rooms, and restaurants, are provided. For the full 118 miles of the turnpike, motorists enjoy uninterrupted travel since there are no cross-roads, no crossing turns, and no traffic lights for its entire distance. Only two stops are necessary; one at the point of entrance to pick up a ticket and the other at the point of exit to pay the toll.

It is expected that the savings in time on the turnpike will range up to 40%, depending on vehicle speeds. Indeed, private motorists travelling on the New Jersey and Pennsylvania turnpikes have been known to average 70 m.p.h. over distances of several hundred miles without risk or fatigue. It is estimated that the accident rate should not exceed 1·5 fatalities per 100,000,000 vehicle miles as compared with the national average of 7·8 fatalities per 100,000,000 vehicle miles, and that these will be due to vehicle defects rather than to travel risks.

These two turnpike roads—surely a monument to American enterprise and vision—therefore provide for the community greatly improved transportation facilities, an absence of congestion through limitation of access and adequate interchanges at traffic intersections, a prospective reduction in accidents to one-fifth of the national average, improved amenities through careful layout and planning, freedom from

strain and anxiety of normal travel, self-sufficiency in finance, without drawing on the national exchequer, and free reversion to the State at the end of 25–35 years, either for abolition of tolls or as a revenue-earning concern.

The attitude of the American Government

It is sometimes said that though the American turnpikes have been successful they are opposed in principle by the Government of the United States, its Highway Departments, and the American motoring organizations. It is true that this method of financing highway development has been criticized, mainly on two points. First, that as it does not have the "faith and credit" nor the taxing power of the State behind it, the Turnpike Authority is obliged to borrow on less advantageous terms than if it had (conversely, of course, this rigorous requirement discourages schemes which are not manifestly self-sufficient and which may run into loss). Secondly, that the funds required should be provided from the general revenues of the State and not be exacted from the individual user. A more balanced and authoritative viewpoint is expressed by the President's Advisory Committee on a National Highway Programme, in its report of January 1955, which says:

"*Toll Roads on Interstate System.*

Some States have utilized the toll method of financing to provide adequate sections on the interstate system. Therefore, our Committee has given careful consideration to this method of financing. As of December 1, 1954, seven States have 988 miles of toll roads in operation which parallel or coincide with the interstate system. The estimated construction cost of these toll roads was $1·1 billion. Another 1,200 miles presently under construction or financed also coincide with the interstate systems. These routes, to cost $1·9 billion upon completion, lie in 9 States, 4 of which have toll roads already in operation.

Agencies have been set up in 17 States and authorized to study and plan nearly 4,000 more miles of toll roads which would coincide with the interstate system. Estimated cost of these authorized toll routes is put at $4·3 billion. However, recent studies disclosed that of the 4,000 miles, at least 914 miles, costing $991 million, do not appear economically feasible.

Thirteen States have proposed, but not yet authorized, another 3,500 miles of toll roads which would coincide with the interstate system. Available estimates set the cost of these proposals at $2·6 billion. Investigations to date on a portion of the 3,500 miles proposed have disclosed that at least 240 miles, costing $200 million, would not be financially feasible.

In summary, 5,242 miles of toll roads in operation, under construction, financed or authorized, either parallel or coincide with the interstate system in 23 States. This milage does not include those proposed projects found not to be feasible. Additional proposals in these States and in five more States, excluding projects found economically unfavourable, bring the total of present and potential toll routes coinciding with the interstate system to 8,527 miles.

Thus, it seems clear that while toll financing on a sound financial basis can meet the needs of a limited portion of the system, it cannot support the cost for the system as a whole. It is obvious, of course, that existing toll roads must be protected in their appeal to traffic.

However, our Committee feels strongly that the Federal Government should not enter into toll-road construction nor provide funds for deficit financing of otherwise non-self-supporting projects. It feels equally strongly that this is a

question to be resolved by State governments. Since the national interest is an adequate highway system, sound toll projects which fit into the system are worthy of consideration by the States, as discussed later in the report.

The Committee believes that major structures such as bridges and tunnels should be financed from tolls to the extent feasible financially. It would leave this determination to the judgements of the States as approved by the Federal Highway Corporation. It does not recommend credit being given for the cost of such structures financed by separate toll charges as compared with lesser structures considered and financed as integral parts of the highway.

About half of the States have provided for meeting their interstate system needs through construction of expressways and freeways of design standards equal or exceeding those of the toll-financed roads, without imposition of tolls, paying for the facilities from current revenues or bond issues of the State amortized principally from gasoline taxes and licence fees. The amount of progress made by this method is about the same as through tolls.

However, neither State nor toll-road financing separately or jointly will suffice to finance the interstate system as it should be constructed and therefore the requisite funds must be found elsewhere. . . ."

". . . Some States have already constructed sections of the interstate system to the required standards with either State or toll financing and others are proceeding along similar lines. Such construction should not be discouraged by this report since our goal is maximum highway improvement. Those States in which sections of the interstate system have been provided to meet the presently established standards for the completed system should receive appropriate credit, provided such funds are used to improve other roads on established Federal-aid systems or as may be approved by the Federal Government and all other Federal funds for highway purposes have been matched as required. No funds should be made available as a credit for toll roads unless the returns from tolls above financing requirements are used exclusively for road construction as contemplated above.

To limit the Federal liability, credit for roads built between 1947 and 1951 should be limited not only to those sections fully meeting the new standards but also to a maximum of 40 per cent of costs other than financing. The credit for those roads completed prior to the calendar year 1955 should be limited to 70 per cent of such costs. In no instance would credit be given for Federal funds expended on the road or for toll roads, in excess of remaining amortization. Roads built at a later date should be credited at full cost.

The funds thus made available to the States will not only encourage matching of available funds but will also make possible accelerated improvement of primary, secondary and other roads, and will encourage local financing of interstate milage to make funds available for other roads without increasing total Federal responsibility. They will be paid to the States only as required to meet the costs of projects approved for construction and it thus appears would provide a major incentive to the highway improvement programme as a whole."

A summary of the characteristics of the major toll roads in the United States is given in Table 10.

TABLE 10.—STATISTICS AND CHARACTERISTICS OF SOME MAJOR TOLL ROADS IN THE UNITED STATES

Toll-road facility	Length in miles	Normal width of median: ft	Number of lanes	Width of lanes: ft	Width of shoulder: ft	Normal width of right-of-way: ft	Intermediate interchanges	Number of structures	Type of paving	Cost of right-of-way and property damage: thousands of dollars	Total cost: millions of dollars	Cost per per mile: thous'ds of dollars	Date of opening
Colorado													
Denver-Boulder . .	17	20	4	12	10	200	1	23	Concrete	510	6·3	371	19 Jan., 1952
Connecticut													
Greenwich–Killingly .	129	8	2† to 8	12	10	180	100	274	Concrete	61,426	398	3,085	Nov. 1957
Florida													
Sunshine State Parkway	104	20	4	12	10	400	6	150	Asphaltic concrete	5,500	74	711	Jan. 1957
Illinois													
Tri-State	80	30	4 to 6	12to13	12	250	28	190	Concrete	27,390	202*	2,525	Not estimated
North Illinois . . .	88	50	4	12to13	12	250	21	158	Concrete	13,478	140*	1,591	Not estimated
East–west	25	50	4	12to13	12	250	8	46	Concrete	2,700	38*	1,520	Not estimated
Indiana													
East-west	156	56	4	12	10	300	9	192	Concrete	9,556	280	1,795	Nov. 1957
Kansas	234	15	4	12	12	300	9	263	Asphaltic concrete and concrete	6,364	140	600	Oct. 1956
Kentucky	40	20	4	12	10	300	4	26	Concrete	2,000	38·5	962	Jan. 1956
Maine													
Original turnpike . .	45	26	4	12	10	300	5	43	Asphaltic concrete	670	20·6	458	13 Dec., 1947
Augusta extension . .	66	26	4	12	10	300	5	90	Asphaltic concrete	1,800	55	830	June 1955
Massachusetts . . .	123	18	4 to 6	12	10	300	12	185	Asphaltic concrete	9,082	239	1,943	Jan. 1957
New Hampshire . . .	15	24	4	12	10	200	1	18	Asphaltic concrete	Not available	7·5	500	25 June, 1950

New Jersey													
Original turnpike	118	26	4 to 6	12	10	300	18	263	Asphaltic concrete	18,500	255	2,161	12 Dec., 1951
Pennsylvania extension	6	26	6	12	10	300	0	Open	Open	Not available	27·2‡	4,500	June 1956
Newark Bay extension	8	6	6	12	Open	250	3	Open	Open	Not available	113·4§	13,800	Apr. 1956
Garden State Parkway	165	36	4 to 6	12	10	200	64	371	Asphaltic concrete and concrete	42,000	285	1,727	Sept. 1954
New York													
Thruway system	562	54	4 to 6	12to13	10	300	Not available	Not available	Concrete	Not available	962	1,712	Jan. 1957
Ohio*													
Project 1	241	56	4	12	10	200	15	368	Concrete	11,223	326	1,353	Oct. 1955
Oklahoma													
Turner turnpike	88	15	4	12	12	200	4	78	Asphaltic concrete	1,285	38	432	16 May, 1953
North-eastern turnpike	88	15	4	12	12	300	5	102	Asphaltic concrete	1,662	68	773	July 1957
Pennsylvania													
Original turnpike	160	10	4	12	10	200	9	156—7 tunnels	Concrete	3,500	76·2	476	1 Oct., 1940
Eastern extension	100	10	4	12	10	200	8	126	Concrete	5,100	87	870	20 Nov., 1950
Western extension	67	10	4	12	10	200	5	98	Concrete	4,040	77·5	1,157	26 Dec., 1951
Delaware River extension	32	10	4	12	10	200	5	78	Concrete	8,500	80·5‡	2,516	Oct. 1954
North-eastern extension	110	4	4	12	10	200	6	153—1 tunnel	Concrete	9,650	217·5	1,977	July 1956
Texas													
Dallas–Fort Worth	30	40	6	12to14	10	300	7	44	Open	8,500	58·5	1,950	July 1957
Washington													
Tacoma–Everett	65	40	4 to 6	12	10	250	30	154	Asphaltic concrete	53,500	194	2,985	Not estimated
West Virginia	88	None	2†	12	9	Not available	4	76—1 tunnel	Concrete	Not available	133	1,511	8 Nov., 1954

* Interest estimated at 3·25%.
† Truck lane on hills.
‡ Including 50% of cost of Delaware River crossing.
§ Mainly urban, including high-level Newark Bay crossing.

Conclusion

British and American attitudes towards the use and construction of turnpike roads present an interesting contrast—a contrast which is perhaps neither inexplicable nor irreconcilable in its nature when the historical and financial backgrounds and, indeed the present political outlook, are taken into account. It would appear that the British attitude is compounded of a dislike arising from memories of traffic delays and often unreasonable exactions for the use of antiquated and outmoded transportation facilities, for which it was felt some more suitable alternative should be provided at public expense, and of a fear that the inception of a toll-road system may be seized upon by the Treasury as a means of evading what are conceived to be its legitimate responsibilities in the matter of providing funds for the construction of new roads.

However, a realistic appreciation of the position in Great Britain must recognize that this point of view is quite impracticable today, and is likely to be so for very many years to come. There is common agreement that the present road system is not only the cause of unnecessary injury and loss of life but also a handicap and expense to all road transport even at its present level.

The toll road is the classic example of the economic self-justification of certain highways, and of the fact that good roads offer advantages which are realized by private and commercial users to exceed the cost of using them. The social, military, and moral obligations of a state or government go beyond this and not all roads can be successful toll roads; but the operation of suitable routes on the toll principle can permit accelerated development of the non-toll-paying part of the highway system, and so extend its benefits far beyond the region to which the toll road itself gives such admirable service.

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Figures 10, 11, 12, 13, 14, and 15 are reproduced by courtesy of the New Jersey Turnpike Authority, U.S.A.

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The Paper, which was received on 19 December, 1955, is accompanied by fourteen photographs and one diagram, from which the half-tone plates and the Figure in the text have been prepared.

Discussion

Mr Gilbert Roberts (Partner, Messrs Freeman Fox and Partners, Consulting Engineers) said that he could speak with considerable feeling as well as experience, because he had spent a considerable part of his working life in designing long-span suspension bridges such as those described in the Paper, which it had been expected would be built but which never had been built and perhaps never would be. In 1928 he had worked for David Anderson on his first proposal for the Forth bridge, and in 1933 on the Humber bridge under Ralph Freeman. In 1945 the Humber bridge project had been taken up again, and in 1947 the Severn bridge was started up with Anderson and Freeman as joint consultants.

No doubt many Members had wondered why, if even half the statements in the Paper were true, those bridges had not been built, and why 5,000 frustrated citizens on one side of an estuary had not got together with 5,000 on the other side and each put down £1,000, making £10,000,000 in all, to build the bridge they needed so much. That, however, would not be enough; even though they had raised the money they could not build the bridge with it unless they had obtained Government permission, since the Government, acting through the Capital Issues Committee or other bodies, that operated restrictive practices, could prevent their spending the money in that way. The proper course would be to approach the Government at the correct time, preferably near the beginning of its term of office, and explain that by giving permission for the bridge to be built it would be doing a useful service to the community, promoting co-ordination, co-operation, and so on, between the two sides of the estuary. The timing would be very important, and the period required for construction had to be reduced. It was easy to imagine a Minister of Transport, who perhaps in any case was not sparkling with enthusiasm for the project, losing interest entirely when he realized that the bridge which he was being asked to inaugurate would be completed by his successor in another Government. The approach should therefore preferably be made in the early days of the new Government and the time of construction described as "not exceeding 4½ years".

Professor A. L. L. Baker (Professor of Concrete Technology, Imperial College of Science and Technology) supported the proposals made in the Paper, in the first place because he well remembered the early days of the work on the Mersey tunnel. He had worked on that project in 1928, at a time when the ratepayers of Liverpool and Birkenhead had had gloomy forebodings about the possibility of the tunnel paying its way. Some enthusiastic gentlemen on Merseyside, however, had seen to it that the scheme had gone through, and the financial results given in the present Paper showed that it had exceeded all expectations. He thought that what had happened in the case of the Mersey tunnel would also happen in similar circumstances in some of the other estuaries where bridges had been proposed and already designed—very handsome bridges, which could well be put in the national shop window in order to help British engineers in competing for bridges overseas.

He felt that engineers were often sounder economists than the professional economists. He remembered that in 1930 or thereabouts, when there had been about 3,000,000

unemployed in Great Britain, the economists had "proved" to the Government that it would not have paid to widen the Great North Road and to carry out other schemes of improvement, but that it would be much cheaper and would help the national economy much more if the unemployed simply drew the dole and did no work at all. Since those days the economists themselves had admitted that their economics had been entirely wrong, and therefore it might be suspected that the economists today who were advising against such schemes as had been proposed might also be wrong in the same way.

It seemed to him to be a basic truth that if construction work was carried out and capital invested in such a way as to cut basic national costs—in the case in point the cost of transport, which was a large part of the cost of goods—the country must gain by it. The Government permitted public subscription for investment in factories which might manufacture any type of goods, including goods sold exclusively inside the country and not exported, so aggravating the present inflationary pressure. If it were permissible by public subscription to invest in a factory, it was surely logical that it should be possible by public subscription to invest in a bridge, which was absolutely certain to contribute to the national economy, because otherwise the venture would not attract subscriptions.

There was also the very important question of maintaining continuity in design offices in Britain. They had all seen the great advantages which the French and the Germans had recently had in tendering in the foreign field in bridge construction. Many of their bridges had been destroyed during the war, and so they had had an opportunity of rebuilding them and of doing research to produce new and more modern types of bridge. They had built up design teams and gained practical experience. Those countries had been able to put forward very low tenders for bridge construction in the countries of the Middle East and in India. He had come across recently some tenders which had been as much as 20 to 30% less than tenders from Great Britain. He attributed that partly to the experience which those countries had gained in rebuilding their own bridges, an experience not available to engineers in Great Britain.

There were other operative factors including the lack of advanced technological education. The White Paper on Technical Education recently issued by the Government seemed to show that the inability of British engineers at times to compete with engineers from other countries might be attributed to the fact that technological education was not so advanced in Great Britain and the Government had not been spending enough money in providing colleges, staff, and equipment. That, however, was an additional cause to the lack of opportunity to gain experience in design in Great Britain during post-war years.

The proposals contained in the Paper seemed to be perfectly sound. He hoped that the Authors' efforts would cause a change of heart on the part of the Government.

Mr H. J. B. Harding (Director, John Mowlem & Co. Ltd, Contractors) said that the Paper, although its theme was economics, struck the imagination very forcibly and showed vision and a nice sense of hard hitting. It was time that somebody did hit hard on those subjects. Dr Glanville had shown courage in his Presidential Address in 1950, and there was need for a bolder approach to all those problems. People in Britain were inclined to think too small, and many members of the Institution also were guilty of that. For instance, in London the only bold conception in the past hundred years had been the Victoria Embankment; the next piece of improvement had been Shaftesbury Avenue, which was little better than a slum.

One day they would have to burst their bonds; at present they were restricted wherever they went, as Mr Roberts had said. When there had been a discussion on high buildings, it had been the outside speakers who raised the best points; their own speakers had dealt with small details like the bending of small columns. Such things needed to be thought about and talked about in a bigger way. If they were bold projects, any objections should be strong and large objections, not little ones.

Great Britain had led the world in the early days in bold projects such as the Forth Bridge, but now there was nothing to show of the same magnitude in Great Britain. There was a need not only for colleges to train people but for work on which they could

gain experience. When engineers had built something of some size it could be shown to those from other countries and thus lead to opportunities for building similarly abroad.

The question of tolls and of possible objections to them was continually being raised, but it should be borne in mind that people did not mind paying for what they wanted. The price of petrol had been doubled, but people bought as much as ever; the price of cigarettes had been doubled and they smoked just as many as before. If toll bridges were built, people would pay the tolls to use them rather than go round.

Mr Harding referred to Road Paper No. 49 [8] on the planning of ring roads round London, which had given Members something to think about. A large suspension bridge of 4,000-ft span really crossed a gap of 8,000 ft, because the total length of the bridge supported by the two columns was nearly double the length of the span, and the cost seemed to work out at about £10,000,000 per mile. The figure quoted in the Paper on ring roads was £123,000,000 for a ring road of 11 miles round London, which was just £11,000,000 per mile, including any subways and tunnels in densely built-up areas and the acquisition of valuable property. The beautiful suspension bridges in Figs 6, 7, and 8 were situated on estuaries and away from towns, but their chief function was to lead to towns, and when the traffic reached the towns there would still be trouble.

Taking London as an example, much of London's congestion could be overcome by having suspension bridges striding boldly across not only London's little river but built-up areas as well. The new Elephant and Castle scheme had been published. Most of the bridges in London led to the Elephant and Castle. A roundabout had been designed which was obviously too small. If the design of the Humber bridge were taken and duplicated, thus effecting a saving in design costs, it could take off from the Post Office at the end of Aldersgate Street and end beyond the Elephant and Castle. The ground underneath would remain fully developed; it would not be necessary to sterilize it and spend £11,000,000 a mile in making a road through built-up areas, because the road would be carried clean over the top. The same method might be used to get to the docks, with a suspension bridge striding across the built-up areas there, instead of using little viaducts. That, however, would not do because it would upset the view of St Paul's Cathedral. Almost everything that could be done in London was turned down on the ground that it would upset the view of St Paul's.

There was no reason at all why such bridges as the Authors had described should not be built. Great Britain lived by exports and must build factories to make the goods for export, but what was the use of building factories if it was not possible to move materials and goods to and from them? Engineers were said not to understand money, but, as Professor Baker had pointed out, nor did economists. The Institution must speak up louder; its members must think bigger and talk bigger and make nuisances of themselves; it was the only way to get things done, having regard to all the opposition and stupidities which surrounded them.

The Secretary of the Institution had called to Mr Harding's attention a verse by Robert Burns which carried its own lesson.

"I'm now arrived—thanks to the gods!—
  Thro' pathways rough and muddy,
A certain sign that making roads
  Is no this people's study.
Altho' I'm not wi' Scripture cram'd
  I'm sure the Bible says
That heedless sinners shall be damn'd
  Unless they mend their ways."

Mr A. L. Somerville (Consulting Engineer), commenting on Mr Morgan's reference to legal delays, said that engineers ought to look on lawyers as they looked on an engineering hazard. Too often planning applications were inadequately presented and the

[8] F. A. Rayfield, "The planning of ring roads, with special reference to London". Proc. Instn Civ. Engrs, vol. 5, Pt II, p. 99 (June 1956).

bureaucrat took advantage of that and merely pigeon-holed them. If the engineer took the trouble to understand the regulations, and even to read them, and submitted the application in the proper form, the bureaucrat could not put it in a pigeon-hole. There was usually a statutory time within which a reply must be given.

The bureaucrat was no new animal. Telford had had twenty-three different turnpike trusts to contend with when he built the Holyhead road.

Mr Morgan had pointed out that the Road Fund was now only a name. Mr Somerville did not think that that had been said with enough emphasis. It had been obvious for many years now that the tax on vehicles was an integral part of the nation's fiscal system, like income tax and death duties and all the other taxes which had first been introduced as a purely temporary measure to meet a particular emergency. It had arrived to stay, and until that fact was realized the public would never enter into an intelligent discussion on the subject of toll roads.

Mr W. T. F. Austin (Civil Engineer, Messrs Freeman Fox & Partners, Consulting Engineers) observed that one of the most important duties of the civil engineer was to provide an adequate system of roads and to maintain them. There would be general agreement that the system of roads in Great Britain was not adequate, and therefore civil engineers as a profession had not carried out their duty. In fact, however, it was not that they had failed to carry out their duty but rather that they had been prevented from doing so because the necessary funds had not been made available to them. Many engineers thought that the provision of the necessary funds was quite outside their responsibility and the job of somebody else. It was true that the allocation of funds for road work was made, and probably rightly so, by politicians, but Mr Austin did not think it was right that the engineer should merely sit back and talk about stresses in slender columns and make designs for bridges and do no more about it. One of the most difficult problems facing the designer of bridges today was finding space to file the designs when he had made them.

It was for the engineer to take steps to see that the necessary money was made available. The politicians had said that the country could not afford it. Mr Austin had made several attempts to find out what the cost to the nation of not having an adequate system of roads was officially estimated to be, but he had not found that there had been any official estimate. The sort of figures which he had been able to find were rough, and he gave them merely as a cock-shy and to give some idea of the order of the cost. It had been said that a reasonable road system would save Great Britain £250,000,000 a year, and he thought that that was of the right order. Motorways would save in the operating cost of vehicles about £25,000/mile/annum. Of the total £250,000,000 savings arising from the removal of traffic congestion, he thought that between £50,000,000 and £100,000,000 would be in London itself, where it was doubtful whether much could be done by means of toll roads, though certainly toll bridges and tunnels might be considered. It should be borne in mind that many of the toll bridges in the United States of America were in the big cities. The Authors had suggested an excellent scheme for providing the money for main trunk roads outside the big cities.

He thought that engineers should endeavour to substantiate figures such as those and should devote far more time in the Institution to discussing Papers which would help them to make better estimates of the benefits which their roads would bring to the country. When they described a road, a tunnel, or a bridge, they should not only say what it would cost but take steps to find out what benefits it would bring and what money, in effect, it would save. That aspect should then be given the maximum publicity. The politicians, who had refused to provide the money to build bridges and roads, were well aware that the general public were getting tired of wasting time in traffic jams and wanted a better road system. They knew that if they were to retain office they had to give the public what it wanted, and they would probably be receptive to sound economic arguments for building some of those roads and bridges within the next year or so.

He had heard it said, even by engineers, that traffic estimates were only guesses.

Traffic forecasting was not an exact science but experience in the U.S.A. had shown that traffic estimates were sufficiently accurate to indicate whether the minimum traffic likely to use a facility such as those described in the Paper would be sufficient to make it pay.

Mr Morgan had pointed out that the New Jersey Turnpike carried twice as much traffic as had been officially estimated. In Table 1 of the Paper the figures given for the traffic generated by bridges in America showed that it generally averaged about 150% of the traffic diverted from existing roads, but it was pointed out that the Americans were in the habit of allowing only for 50% in their estimates. That was one explanation of why traffic on the New Jersey Turnpike exceeded the estimate of the *minimum* to be expected. The people who made the estimate no doubt thought that it would, but the people who put up the money had to be convinced that the traffic estimates were on the conservative side, and on the whole, in the examples given in the Paper, they were.

If the columns in Table 2 were added up it would be found that less than half the net income of the bridge was actually paid for the construction and design of the bridge; about three-fifths went as interest on capital. The example had been drawn up in accordance with the principles in the booklet on economics issued for students.* Whatever was going to be done to put right the road system of Great Britain, however, would take many years, and the period would exceed the period of a normal loan. Mr Austin considered, therefore, that there was no point in financing such schemes by means of loans. He suggested that the interests of the nation would best be served if the Government were to make certain relatively small grants for a period of a few years which would make it possible to build a few structures which were likely to pay well. Those would be grants made out of taxation and would carry no interest. If a bridge costing £15,000,000 were built over a period of 5 years it would require £3,000,000 a year, and when it was opened to traffic it would have been paid for. Tolls could then be charged to those who used it, and those tolls would provide the money to build the next bridge. According to the Tables in the Paper, after 5 years of operation as a toll bridge it would be possible to start construction of the next, and after a further 5 years the second bridge would be complete. The first bridge could then be freed of toll and tolls would then be collected from the users of the second bridge, and so on.

Mr Edward Ogden (Senior Assistant to County Surveyor of Lancashire) remarked that it was no good saying to the R.A.C. and the A.A. that the Road Fund had gone. They were very conscious of the fact that their members paid a great deal in taxes. In Great Britain the taxes paid by motorists amounted to four or five times the expenditure on the roads, whereas in the United States of America those taxes did not cover the cost of construction and maintenance of the roads. There was an obvious reason, therefore, for the outlook of the British motorist differing from that of the American.

The Authors had suggested that it was not possible to expect to meet the cost of constructing new road facilities out of revenue. The programme that had been put forward recently by the Ministry of Transport for the construction of new roads had been estimated to cost £147,000,000, and that was to be spread over a number of years. That was a miserably small proportion of the taxes of £420,000,000 a year paid by motorists.

It seemed to him that the criterion was whether national resources should be devoted to the construction of new roads. It had to be borne in mind that it was cheaper to construct a free road, on which tolls were not paid, than a toll road. In one example given in the Paper it had been stated that the cost of collecting tolls was 20% of the running cost. It required a number of men to collect the tolls. On the New York Thruway system there were 480 toll collectors in 366 miles. If there was a motorway running from London to Glasgow, in Lancashire the intersections with that motorway would occur twice as frequently as they did on the New York Thruway, so that more toll collectors would be required. It would require at least 500 men to collect tolls on a toll road from London to

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* "An Introduction to Engineering Economics for Civil Engineering Students." Instn Civ. Engrs, 2nd Edn, 1956.

Glasgow. So far as the nation was concerned they would be non-productive, and he felt that their employment was nugatory.

Apart from the collection of tolls, there was the extra capital cost involved in providing collecting facilities. At present it was possible to design an intersection and run straight up from the existing road to the motorway in about 100 yd. If there were toll facilities to be provided it would be necessary to bring all those side roads together or into two points and to provide standing areas for the motorists, so that the cost of collecting tolls was also going to put up the cost of the road.

A very important point was that if national resources were to be devoted to producing a road, it was essential that the maximum amount of traffic should use it and profit from the advantages which would accrue. In particular, the more traffic diverted to a motorway from built-up areas, the greater would be the reduction in the appalling casualties which at present occurred there.

Similarly, the savings in operating costs and time should be as great as possible both from the users' and the national viewpoint.

The Authors had demonstrated clearly that toll bridges and roads were an economic proposition. An even better economic proposition would be roads which did not have to meet the cost of collecting tolls and which carried more traffic than even the Authors envisaged.

Mr G. A. Wilson (Chief Engineer, Port of London Authority) said that so far only one side had been heard, all the speakers with one exception being interested parties—contractors, consultants, and county engineers. He would have difficulty in giving the opposite view because he sympathized with the Authors. Mr Ogden from Lancashire had given a local view and had shown some of the difficulties which had to be faced. Mr Ogden had left the matter with the Government, but the discussion seemed to show that the Government methods were too slow.

Mr Morgan had said that the crux of the matter was finance, and in Part 2 of the Paper the Authors had stated that only 1% of the production of steel in Great Britain would be required. Mr Wilson did not think that that was the right way to forward the adoption of any engineering project. To argue that only a small demand would be made on resources was not so effective as demonstrating that the cost of deferring the work could not be afforded.

Owing to parsimony in the past the present wasteful situation had arisen and action was needed to prevent it persisting. Mr Wilson believed that a recent Minister of Transport had said that the private financing of toll projects would be acceptable but no proposals had been made. Perhaps some Ministerial encouragement was needed.

The programme set out in the Paper was about 700 miles of road at £300,000 per mile, which meant finding about £250 million. Mention had been made of possible savings, but the Authors had not set out to show, for example, how much fuel could be saved. Fuel saved would reduce imports and, if it were possible to reduce imports by carrying out such a programme, that would be an argument which could not be easily ignored.

A good deal had been said about economics. The Chancellor of the Exchequer had stated recently that the country was spending too much. As Mr Wilson understood the matter, Great Britain had a certain earning capacity, and the annual earnings were spent by Government Departments and by private enterprise. At the present time those two agencies were spending too much, and the spending of one or both had to be cut down. With Government expenditure it was difficult to know when a project was marginal. In so far as private enterprise took up those projects, the Chancellor of the Exchequer would be relieved of that doubt, because a private enterprise project would not be supported unless it could be shown that it would pay its way. Every economic private enterprise project which the Chancellor of the Exchequer permitted would reduce the amount of money on the market looking for employment. The Chancellor of the Exchequer would get an asset which would pay for itself and increase the national earning capacity, and in so far as he approved those private enterprise projects, he should be able

to reduce Government expenditure on roads and bridges. It seemed to Mr Wilson that that argument could be developed on those lines to support the Authors' case.

Other countries had developed their road systems as a military necessity for the movement of troops. It had been stated that in the next emergency it would be necessary to disperse the population, and it seemed to him that there might be some case from the point of view of civil defence to support those projects for toll roads and bridges.

At the end of the Paper there was a splendid series of new bridge projects which rose rather like sacred tombstones. Mr Wilson felt that he must add one more memorial to the crowded shrine. A tunnel under the Thames between Dartford and Purfleet was proposed and it was also suggested that a barrier should be placed in the same area to prevent the flooding of London. It seemed to him that a two-track tunnel was an inadequate traffic provision across the Thames, particularly in view of the figures which had been given for the development of traffic. A barrier across the Thames could be designed to operate as a road bridge when not required to combat a surge. Such a structure might rank in size with some of the projects in the Paper and it remained to be seen if it would become just another memorial to the neglect of communications which was impeding British trade.

Dr H. Q. Golder (Director, Soil Mechanics Ltd) said that the main problem involved was not an engineering problem and not even a financial problem; it was a political problem. In Part 1 of the Paper, on p. 578, Mr Morgan had given a list of seven requirements, of which the first was finance. There would be no argument about the fifth, which was technical knowledge; it would be generally agreed that there was plenty of that available. The second, third, and fourth were the important items; if they were lacking, Britain could accept a loan in gold from Russia to provide the finance, but would not get any roads. It was essential to have manpower, plant, and materials. The Government would say that that would mean a diversion of those things from their employment elsewhere. The argument would be that Great Britain could have atom bombs, or a health service, or roads, but not all three.

It was only necessary to look back to the war years to see that that was nonsense. During the war Britain spent an enormous sum of money on munitions, which she then exported freely to the enemy. It had an enormous health problem, which had been fairly adequately dealt with. It had not built any roads at that time, but, as was mentioned in the Paper, it had built aerodromes which were equivalent to hundreds of miles of double trunk road.

He did not think that there could be any question that Great Britain could have the roads if it had the finance. Two methods of financing the roads were apparently available. One was the normal method of Government finance, presumably by Treasury bonds or something of that sort. The second method was that of private finance, somebody else putting up the bonds and the public buying them. Not many people had money in the bank to go out and buy them, but they would say "That sounds a good buy," and they might buy £100 worth. The Authors had shown that a profit would be made. Towards the end of the year those people might find themselves short of money and have a slightly larger overdraft than usual. The net result was that as in the case of Government bonds the roads were paid for by the creation of credit or increase in debt, and that was the normal and natural way in which all big projects were financed. Anyone who did not believe that had only to look at the way in which the national debt had increased since it was created in 1694, when the Bank of England had been given the right to lend £1,200,000 to the King.

The only problem remaining was a political one—how to make somebody do something about it.

Mr A. J. H. Clayton (Divisional Engineer, Parliamentary, Improvements, and Town Planning Division, Chief Engineer's Department, London County Council) observed that it took a long time to get a project adopted in Great Britain. He believed that the

Victoria Embankment had been originally proposed about 1800. Work was about to start on the Notting Hill Gate widening—the old toll-gate bottle-neck. The London County Council had first applied to the President of the Board of Trade, before there was a Minister of Transport, for a grant for that project in 1910, and it looked as though they were going to get it before the end of the present financial year!

There was nothing new about tolls. Many of London's bridges had been built by tolls. A hundred years ago there had been some toll bridges and some free bridges over the Thames. The free bridges had been very crowded, even though the toll for pedestrians had been only ½*d*, because people preferred to walk a long way round and would use Westminster bridge rather than pay ½*d* to cross by Waterloo bridge. Whether motorists would adopt the same attitude today he did not know.

In putting forward a very good case, however, it was important not to spoil it by arguments which could be criticized. The rates which had been used to support the argument that people would pay for toll roads put the value of time and distance rather higher than he thought the motorist would put it. In Table 4 the figure for the ordinary car was put at 6*d* per mile. Taking everything into account, that probably was the overall cost but when a motorist was thinking whether he would make a detour or cross by the new bridge he would be inclined to think in terms of the 2*d* per mile which he paid for his petrol. The figures given by the Authors were not essentially dissimilar to those given by Mr Rayfield in his Paper for estimating the value of those projects to the community, but Mr Clayton did not think it was reasonable to take those figures as representing the value which the motorist himself would put upon it when he was asked to pay a toll.

It was important that in the design of toll roads and their siting the traffic-design aspect should be considered. It was mentioned in the Paper that the dockside branches of the Mersey road tunnel had to be closed at the busiest time of the day. Traffic engineers had known before the tunnel had been built that the intersections would cause trouble. Such matters were often decided on the basis of factors other than careful engineering design.

Although not positively stated it seemed to be taken for granted in the Paper that the Dartford tunnel would relieve traffic in London. Mr Clayton thought that the relief would be negligible. It would give some small relief to Blackwall tunnel, but that would be taken up by those who wanted to use Blackwall tunnel in any case. The estimated traffic in the Dartford tunnel would be less than that still using Blackwall tunnel, with its 16-ft carriageway and sharp bends. From the broad economic point of view of the country as a whole it would have been better to duplicate Blackwall tunnel. Those questions ought to be looked at from the traffic-engineering point of view, as well as from the structural-engineering point of view.

Mr Clayton was surprised to hear a speaker suggest, apparently without any factual basis, that the proposed roundabout at the Elephant and Castle was too small and that what was wanted was a suspension bridge across Central London from north to south That would be a very costly white elephant because drivers wanted to go to various places in inner London and not across to the other side.

Mr C. D. Morgan (Secretary, British Road Federation, Ltd) remarked that the synopsis of the Paper stated that "The purpose of this Paper is to demonstrate the possibility of building some of the badly needed long-span road bridges and tunnels in Great Britain as financially self-liquidating projects." He thought that it was generally accepted among road users that that was the only way in which those expensive and short-cut projects were ever likely to be built, and if only for that reason, he did not think it would be found that the majority of operators would be opposed to tolls on that sort of structure, i.e., bridges and tunnels. The synopsis, however, went on to deal with the road section of the Paper, and the problem there was entirely different. Whilst agreeing with Mr Harding's desire to think big, it would be necessary also to think carefully about exactly what should be done to finance new highway construction.

Undoubtedly it delighted the engineer to build large structures and to build roads which

strode across the countryside, but such things were not built solely from an engineering point of view. New highways were urgently needed to improve the capacity of the national economy and to reduce costs. Those objectives would not be achieved by building new roads which would cost the operator as much as he had had to pay when using the old ones. So far as the road section of the Paper was concerned, there was perhaps a gap in the facts as they were presented. Why was it necessary, for example, to look only at conditions in the United States of America? Great Britain wanted good roads, and had to think big. The nation's world market competitors were in Europe, and especially in Western Germany; there, and in Holland, Belgium, and other countries good free motorways were already in existence.

Mr Morgan suggested that so far as the Institution and road operators were concerned, it had to be recognized that new roads would not be financed from the Road Fund that had ceased to exist even in name. Nor was the time which it took to collect tolls a valid objection against toll roads because American experience was that it could be done very quickly. The whole argument was an economic one, and he hoped that before the discussion closed some reference would be made to the other possibilities for building motorways and express-way types of road, which were going to be expensive. So far only very minor schemes had been considered in Great Britain. It was difficult to believe, for example, that the Notting Hill Gate scheme would make any significant change in the traffic problem of London. Something might be done, as Mr Harding had suggested, by a Humber bridge three times multiplied, striding across the built-up areas. Such schemes were bound to be very expensive, and it was a question of obtaining both public and Government acceptance for an adequate part of the national resources to be devoted to those purposes.

In saying that there must be public acceptance of the need to use national funds for that purpose, it should be pointed out that there was a vast difference between Britain and the United States of America. In the United States there was a State and Federal petrol tax of 7 cents, with very low vehicle taxes, and 95% of the revenue from those taxes went straight back into road construction and repairs by the Federal and State Governments, so that the road user in America had almost the whole of his road taxes returned, but was still not satisfied. Where the very successful toll projects had been built in America it had been a question of the State concerned collecting money from through traffic, and it had been seen that there was a good revenue to be derived from that through traffic. In Great Britain, on the other hand, there was no need to point out the level of motor taxation and the proportion of the money which went back into the roads. He agreed with the necessity to think big, but it was important to avoid confusion of thought on the financial and economic aspects of the whole road problem in Great Britain.

Mr D. M. Brancher (Engineering Assistant, Worcestershire County Council) said that it seemed that in Great Britain there was an organized body of thought behind the demand for better roads, and that was a good thing, but that body comprised both those who wanted better roads at any price, provided the country needed them, and those who wanted better roads but who were tired of the raiding of the Road Fund and wanted to see their operating costs reduced. There might be a split in the ranks of those people if the ideas proposed that evening were put forward. That should not really be a cause for anxiety, because those ideas might well capture the imagination of the people of Britain, who had already had experience of being excited by the prospect of big civil engineering works. The present state of affairs, where there were no votes in roads, would come to an end if people began to realize that the roads formed a very important part of their living costs and of their whole standard of living.

Mr Austin had suggested that instead of charging tolls to meet loan charges, the money derived from tolls should be put on one side to finance the next bridge. Anyone who had looked at the history of the Road Fund must view that prospect with a great deal of anxiety.

Dr W. H. Glanville (Director of Road Research, D.S.I.R.) said that he had been

particularly struck by Dr Golder's contribution, because he thought that Dr Golder had been quite right; it was a political problem. Great Britain was not yet fully alive to the importance of the problem, and until the people were prepared to insist that the Government should set to work to solve it, little of consequence would happen. In America, there was one car to every 2·9 people, and the Americans really believed in roads. That was why they got them built; they all wanted them. Until the people of Great Britain really wanted them, they would not be built. That was the problem.

He was convinced that if one of the proposed roads—whether a toll road or an ordinary motorway—was built and began operating it would suffice to prime the pump. The people of Britain had not seen the type of road which could be built. It was all very well showing them pictures, but they had not seen for themselves what it meant to have a proper motorway on which they could travel rapidly from one centre to another, so that they did not appreciate what it meant. If one could be built, whether from London to Birmingham or somewhere else, and people had the opportunity of experiencing the advantages of travelling on it, the position would, he felt sure, change rapidly.

The problem was how to get that one road built, and that was where, to some extent, the economic arguments which the Authors had put forward would help. He was not sure, however, that something very much more forthright was not required, some much plainer speaking of the kind which they had heard from Mr Shirley Smith, to generate a proper appreciation of the problem by the people of Britain.

The economic problem was not a simple one. It was not easy to make a proper economic study of one of those projects. The Road Research Laboratory was now engaged on an economic study in connexion with the London-Birmingham motorway. They were now measuring the traffic, making studies of its origin and destination on the roads leading from London to the north and north-west, assessing fuel costs, operating costs, and times of journeys. They hoped to be able to make a reasonable estimate of what the motorway would carry. It seemed clear, from what they had done so far, that it would in fact have to carry a very considerable amount of traffic indeed, if it were to satisfy the demands which would be made on it. The problem was tedious rather than difficult.

The attitude in the United States of America towards toll facilities had changed somewhat in recent years and the motoring associations did not seem to be so much against them as they had been at one time. In fact the present attitude was that although they would much prefer to have roads and bridges without tolls, if they could only have them with tolls then they would have them just the same. There was no reason to suppose, in his opinion, that the attitude of the motorist himself would be very different in Great Britain. He would far rather have the cost spread over the community but if toll facilities were unavoidable he would rather have them than go without a facility by which he would profit more from the saving in time or money or both, than he would be out of pocket by the toll charge.

He regarded it as most important that such discussions should take place and that engineers should, as Mr Harding had said, adopt a bolder approach to those matters. He only hoped that they could find a second Telford, someone with Telford's character and influence, to carry their plans through. They needed someone who would lead the country to a better appreciation of what roads could do and of what roads were really needed.

Mr J. K. Anderson (Partner, Messrs Mott, Hay, and Anderson, Consulting Engineers) said he spoke as another of those consulting engineers whose staff was filing drawings away in drawers which were becoming full of plans for the Forth road bridge, the Severn road bridge, and so on. It was giving them a kind of engineering indigestion. He did not know what it would develop into eventually, but they badly needed a pill or two to put that indigestion right. At the moment they were looking to the Ministry of Transport to provide those pills and he hoped that something would soon be done about it. As Dr Glanville had suggested, there was need for the completion of one scheme at any rate to make it possible to go ahead with others.

Mr Anderson had visited America some years ago and had seen the New Jersey Turn

pike under construction. In June 1951, he had watched work in progress on some earthworks, where bulldozers were still being used for consolidation and there had been no sign of any surface, yet in November part of the project at any rate had been opened. That indicated the great speed at which such projects went ahead, and that had been one of the things which had struck him most in America. He saw no reason at all why British engineers should not do the same and get the work done quickly and efficiently, enabling a return to be obtained more quickly. He hoped that the people who were going to provide what he had referred to as "the pills" would consider all those matters.

He had noted that the loading of American structures was rather less severe than the loading of structures in Britain. He had been looking up the details of one structure where provision was made only for 10-ton axles. It would be much easier in Britain were it only necessary to provide for loads as light as that. The loading gauge in America was also rather less, being, he believed, 14 ft against 16 ft 6 in. in Britain. The American standards, if adopted here, would make for economy in the works which were proposed.

In considering toll structures, the economics of them deserved a great deal of thought. There was one economic aspect which had not been touched on so far. Dr Glanville had referred in his Presidential Address to the great saving which would take place so far as accidents were concerned, with the avoidance of loss of manpower and with less cost to the health service as a result of the smaller number of fatalities and accidents on the roads. Mr Anderson felt that that was a very important point when considering the economics of the subject, and was in favour of heavy expenditure for the improvement of the roads.

It had been suggested that if such a toll structure was built a certain income would be derived from it. He would suggest that a very good income might be obtained by building many such toll structures. It would certainly be a more profitable source of income than doing nothing at all. He sometimes wondered whether it would be possible for a syndicate of engineers to get together and go ahead with some of those projects, possibly by getting an Act of Parliament for a specific structure, raising the money, and going ahead quite independently. Legally it would be possible to do that if such a syndicate or such syndicates could be formed. He hoped that great publicity would be given to the Paper and to the great possibilities which existed for the toll structures and turnpike roads to which the Authors referred. He would support any suggestion for publicity and for bringing pressure to bear on the authorities.

Mr H. L. E. Holland said that he had visited the United States of America in 1951, to study American highway methods. Although mostly concerned with the more strictly technical aspects of road and bridge construction he had had an opportunity to learn of the ways and means by which extensive highway works had been financed and constructed and had been received by motorists in many parts of that country.

It should be recognized that during the past 60 years of economic development in the United States, the automobile had been looked upon as an essential to social well-being as well as in business. Consequently the development of roads had been a subject more closely the concern of the individual, and although in America there was an extensive railway organization, there had never been the same public reliance on the railways as the prime means of transport and communication as in Britain. A contributory factor, to mention one only—was that America during the past 60 years had been able to find within her own borders a considerable supply of oil which had added its own impetus to highway development. It should not be thought that the citizens of the United States were any more satisfied with their roads even today after more than 10 years of intensive post-war construction, than were the people of Great Britain. Nor was it the case that all of the modern multi-lane limited-access roads built in the United States since the war were toll roads. In California for example, and particularly in Los Angeles, there had been built in recent years an extensive network of "freeways", paid for out of Federal, State, and City Funds, which were not subject to tolls. In California it had been demonstrated that there was a sharp difference of opinion on the question of toll roads versus freeways, and he believed that elsewhere in America the same was true. In general, wherever a toll road had been constructed in the States there was already in existence an alternative

route. Such alternative routes remained the responsibility of the Federal or State Highway Authority for maintenance purposes and were kept at a high efficiency level. A well-known example was the Lincoln Highway, which he believed was designated Route U.S.40, and which ran east-west across Pennsylvania alongside the famous Turnpike, often crossing and re-crossing it during the entire length of about 300–400 miles. It was not a divided highway, nor were its alignment and general standard of construction up to the standard of the Pennsylvania Turnpike.

On one occasion whilst in America Mr Holland had been told of the proprietor of a small trucking concern, who had started using the Turnpike when it was opened to traffic, but who, after a short while, had routed his trucks again by way of U.S.40. One reason given was that his drivers had been involving him in losses when using the Turnpike—due to speeding, and to relaxing their attention on the easier journey, leading to more frequent accidents.

In Britain, where the railway network, had already been spread firmly over the country before the advent of the motor-car, and where indigenous coal had long been the common fuel, it was not difficult to appreciate why road development never had received the same impetus as in America.

Industry in post-war Britain was geared to the need to export in order to obtain from overseas food and essential raw materials—notably oil. In view of the resulting shortage of certain materials for use at home, and under the planned economy to which there was no alternative but to submit, there had to be a central authority to determine priorities. That authority could only be the Government, deriving its power from Parliament elected by the road users. Granted that there was the technical knowledge and contracting plant available and a willingness to subscribe to a bond issue to finance a toll bridge or road, there was still the problem of finding the requisite materials for the facility proposed.

Therefore he believed that at present—and until the economic position improved—the Government could not be expected to authorize several isolated toll projects, each of an estimated value of 10 to 15 million pounds sterling. It was preferable first for the Government to pronounce in favour of the principle of charging tolls for the use of and to finance such projects, then to demonstrate what could be done. A relatively small but important bridge, strategically situated, might be built, either by means of a bond issue or financed directly by the Government. Such a bridge should offer a short cut and a means of avoiding a congested area for the payment of a toll, estimated on the basis of the time and distance saved to the user. It should not be difficult to determine a suitable location for such a pilot scheme costing 1 or 2 million pounds. An example which occurred to him was the high-level bridge over the mouth of the river Neath, which formed part of the recently completed Neath by-pass scheme in South Wales. That bridge reduced the distance by road from Port Talbot to Swansea (previously about 13 miles) by 5 or 6 miles.

*** **Mr J. F. Pain** (Manager, Bridge Department, Dorman Long and Co.) observed that the advantages of the improved amenities described in the Paper were obvious, and that the lack of them was economically indefensible. Upon that score the Authors were preaching to the converted. They had pointed out a well-tried method by which those advantages might be attained and had sought to show that a handsome profit awaited the promoters. Still nothing was done!

On general grounds the provision of an adequate trunk highway system should clearly be a national responsibility, but in both Britain and America the failure to meet that responsibility had led to a demand for an alternative and the toll principle offered a solution. Exasperation with the present state of affairs and the obvious fact that nothing would be done to remedy it were the main justification for the proposal. To judge by the success of the Mersey tunnel, public prejudice against the toll principle evaporated in practice, whilst in America it had been an unqualified success.

*** This contribution was submitted in writing after the closure of the oral discussion.—Sec.

Politically, the sponsoring of major public works was a profitless undertaking owing to the time required for their construction. There was too great a risk that an opponent might be accorded the credit on completion. The handling of the road programme over the past 30 years provided all the proof necessary. Without extreme pressure from public opinion nothing would be done. A device was essential whereby the responsibility for some, at least, of the most urgent works was transferred to a body which would regard them more objectively.

Since the primary responsibility for a proper highway system still rested necessarily with the Government, it seemed that the toll principle should be involved in only a limited number of the most urgent cases (of which there were many) as a sort of "primer". The conditions in Great Britain differed so widely from those of America, particularly with regard to the density of minor roads and rail communications, that the application of the principle to road construction appeared a doubtful expedient.

Only the most formidable pressure of public opinion would bring a Toll Authority into being at all. In that, civil engineers would appear to bear a heavy collective responsibility to draw the country's attention to the terrible cost of present deficiencies and to the relatively painless manner in which some at least could be remedied by the creation of a Toll Authority. If the collective interest of industry and the motoring public, as the greatest sufferers from the present state of affairs, could be aroused and adequate publicity obtained, something might be done. There was little doubt that one success would generate others and ultimately force some national action.

Mr Morgan, in reply, said he felt that Mr Somerville's solution of the problem of legal delays was an over-simplification. The minds of bureaucrats were very fertile of ideas tending to delay. If pigeon-holing failed, they thought of something else, such as devising a multitude of forms which it was quite impossible to fill in. It was true that the bureaucrat was no new animal; he was simply another pest which it had not been found possible to exterminate.

Mr Morgan agreed that a great effort should be made to make the public understand that the proposed toll roads would be of great value to the country if used by traffic for which the value of the time saved would be immeasurably greater than the cost of the toll. That would, at the same time, relieve the remaining roads of traffic and improve conditions for other traffic.

Mr Ogden's remarks appeared to beg the question. It was perfectly obvious that the so-called "road fund" had now become simply a means of raising taxation and, in Mr Morgan's opinion, should be welcomed by every income-tax payer, since it spread the burden over the whole community. Everyone who paid for a ticket on a motor omnibus helped. Mr Morgan was not impressed by Mr Ogden's argument that a large number of collectors would be required on toll roads. If he were to make an estimate of the number of man-hours saved by each of those men, Mr Morgan felt sure that he would not regard them as parasites.

Mr Wilson had suggested that speakers in the discussion were, with one exception, interested parties. Mr Morgan's impression was that Mr Wilson had not made that remark very seriously and therefore felt that he could, in the same spirit, point out that Mr Wilson had been guilty of special pleading later on in his remarks when he referred to the advantages of a Thames barrage as a part-time road crossing.

Mr Wilson had said that he felt that assessment of the demand on national resources was not the best way to put forward the adoption of an engineering project. Mr Morgan had said that the crux of the matter was finance but the other matters had been mentioned in order to forestall the old arguments which had been produced in the past with monotonous regularity to the effect that even if we could find the money, we had not the man-power, the steel, or the cement. In view of past history the Authors felt that all those points should be dealt with in their turn.

In reply to Mr Clayton, Mr Morgan pointed out that it was quite unrealistic to compare the cost of a toll bridge or tunnel with the cost in petrol of going round the long way. That left out of account the saving of time, the value of which would usually much exceed

those costs. The road user was not obliged to use a toll route; he could go by the alternative route if he liked. Road users, however, should realize by now that the choice was between toll bridges and tunnels or nothing at all.

Mr Bartlett and **Mr Shirley Smith** did not think that any points had been raised which called for a reply. The unanimous endorsement given to the Authors' proposals for the promotion of long-span bridges and tunnels as financially self-liquidating projects was most encouraging.

They would like to make it clear that they had no wish to enter into political controversy as to the best way of spending public funds. Their intention in the Paper had been simply to present the economic and technical aspects of the matter and assemble the facts so that they would be readily available for any engineers or public authorities who could make use of them. They were glad to record that the Institution had taken prompt action to ensure that the proposals in the Paper and the views of members of the Institution had been conveyed to the authorities concerned.

The Authors agreed with the opinion of a number of speakers that it only required one or two such toll projects to be built in the United Kingdom, whether bridge, tunnel, or motor highway, for the public to be swiftly converted from the attitude of doubters to that of enthusiastic users, clamouring for more.

Since the Paper had been presented some important developments had occurred which gave cause for satisfaction, and one might even say, optimism. A contract had been placed for the construction of the Dartford tunnel, work on which would proceed at once. The Minister of Transport and Civil Aviation had also stated on the occasion of the Annual Dinner of the Institution that he intended, if possible, to see work started during his term of office on one of the badly-needed long-span highway bridges in Britain. Furthermore, he had made it clear that the users of both those facilities would have to contribute to their cost by paying tolls. That was an immense step forward!

Mr Richards, in reply, said that with such general agreement with the main theme of the Paper, neither clarification nor argument seemed to be required and conviction might follow if the Authors reiterated that they had not sought to produce a case for the construction of toll roads, bridges, or tunnels, as it were *in vacuo*, nor to prove that they were better than free facilities where such could be provided. They had merely sought to show, and believed they had demonstrated, that adequate traffic facilities were economically self-justifying and that where free highways could not be provided there was a sufficient basis for building and financing them as self-liquidating projects.

The Authors had not been able to bring themselves to share the optimism of those who, contrary to the experience of more than 30 years, believed that an active road-construction programme, adequate to the needs of Britain, was just around the corner, nor the complacency of those who believed in waiting until it was; and it was in the belief that an adequate road transportation system was a plain necessity in the economic welfare of the country, that it would, in appropriate conditions be self-justifying and yield a dividend at least as great as any other form of capital investment, and that in satifying those conditions there could be a general benefit to the highway system as a whole, that they had put forward the arguments contained in the Paper.

Correspondence on this Paper is now closed.—Sec.

PUBLIC HEALTH DIVISION MEETING

arranged in conjunction with the British Nuclear Energy Conference

12 April, 1956

Mr W. A. M. Allan, Member, in the Chair

The following Paper was presented for discussion and, on the motion of the Chairman, the thanks of the Division and of the member Institutions comprising the British Nuclear Energy Conference were accorded to the Authors.

Public Health Paper No. 15

THE CONTROL, CONVEYANCE, TREATMENT, AND DISPOSAL OF RADIOACTIVE EFFLUENTS FROM THE ATOMIC WEAPONS RESEARCH ESTABLISHMENT, ALDERMASTON

by

* William Lawrence Wilson, O.B.E., B.Sc.(Eng.), A.M.I.C.E., M.I.Mech.E.,

Percival Albert Frederick White, B.Sc., M.I.Chem.E., and

John George Milton, M.I.Mech.E.

SYNOPSIS

The Paper states that the principles adopted were developed from those originally devised for and used at the Atomic Energy Research Establishment, Harwell, and shows how attempts were made to overcome certain limitations that became evident there in the construction and early operational phases. It sketches these limitations and describes the solutions adopted.

The Paper mentions the prospective use of special decontaminants and the measures taken to deal with them.

A description is given of the research that was undertaken to determine the form of the effluent-treatment and distillation plant and shows the approach which led to the type of plant and its required performance. The types and costs of chemical additives examined and the final results achieved are stated.

The full-scale plant is described in detail together with particular additional developments, primarily those dealing with the safe "canning" of active sludges.

The investigation into disposal points for liquid wastes is outlined and is followed by a description of the storage facilities and discharge pipelines. Attention is drawn to the steps that were taken to convey the effluent safely.

A description of the works at and in the River Thames is given together with an outline of the special trials showing how diffusion therein was achieved.

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Statement of the problem

The problem was for all liquid radioactive wastes to be collected at the point of origin, segregated into varying qualities, contained and controlled to prevent contamination of people and plant, examined to determine quality, purified to a degree where they would not be harmful to animal and plant life, and then discharged to the nearest practical disposal point. All solid wastes arising from the foregoing were to be collected and "canned" by a safe process for ultimate disposal to sea.

Data

Quantities

(1) Process wastes to be determined by laboratory experiment.
(2) Laboratory wastes to be assessed from current practice at Harwell.
(3) Wastes arising from decontamination processes to be assessed.

Possible means of disposal

To sea by tanker or by barge through the Kennet-Avon canal; to sea through existing "Pluto" pipelines near the site; or to Rivers Kennet or Thames.

Tolerance concentrations

Maximum permissible concentration additional to rivers or other potable water sources used as a channel for disposal to be as follows:—

Alpha emitters

Radium: 4×10^{-10} microcuries/cc.
Other alpha emitters: $2{\cdot}4 \times 10^{-9}$ microcuries/cc.

Beta emitters

Radio calcium and radio strontium: 2×10^{-8} microcuries/cc.
Other beta emitters: 10^{-6} ,, ,,

Key and Kenny's Paper[1] enlarges on this subject and emphasizes the minute quantities involved.

Control

The plant provided to give the tolerance concentrations quoted above; these to be subject to continual check by the Ministry of Health, Thames Conservancy, and Metropolitan Water Board.

Historical

Two Papers[2, 3] were read recently on an analagous subject to this as applied to the Atomic Energy Research Establishment (A.E.R.E.), Harwell, the first operative atomic establishment in the United Kingdom. In those Papers it was recorded that the principles adopted were obtained from an examination of probabilities and possibilities rather than from a technical analysis derived from known data.

The wide variety of the work to be undertaken at Harwell necessitated treatment plant of a flexible character which could be expanded or modified to suit developments. For this station the basic principles of effluent control were enunciated, namely:—

(1) Segregation.
(2) Concentration.

[1] The references are given on p. 655

(3) Local examination.
(4) Bulk storage.
(5) Treatment.
(6) Post-treatment bulk storage.
(7) Dilution.
(8) Discharge.

When the Aldermaston project was entrusted to Ministry of Works executive staff in early 1949 these principles had been under trial for a relatively short time. In the limited operational time of about a year the results were encouraging enough to justify continuing on these lines. The initial knowledge that at Aldermaston relatively few radioactive materials would appear in the effluent and that these would be predominantly alpha emitters arising from processes novel but likely to be practical, gave the problem a sharper definition than was previously experienced.

Consequently, the Aldermaston works caused few original misgivings, so far as basic design was concerned. The time element presented the most onerous problem, since the project was linked with the later and original events at Monte Bello, which were timed for what seemed to be an incredibly short period ahead. However, the installation was provided successfully and in time to permit the Monte Bello trials to proceed to schedule.

MODIFICATIONS ARISING FROM EXPERIENCE AT HARWELL

Although Harwell had been in operation for a relatively short time when designs for Aldermaston commenced, experience showed that modifications were desirable. These are listed below.

Number of effluent systems

At Harwell there are three effluent systems: (*a*) the radiochemical drain; (*b*) the so-called clean-process drain; and (*c*) the soil-and-surface-water system. The last two of these were not provided with control facilities. It was thought originally that only the first would become contaminated with activity but soon all systems were found to be affected.

Contamination of (*b*) and (*c*) was very limited in extent. Nevertheless it existed and seemed to show that on a site comprising about a hundred buildings and employing thousands of men, contamination could not be avoided at all times. Harwell promptly modified the system provided to permit the monitoring and control of all wastes. Consequently, it was decided that at Aldermaston the radioactive area (only a relatively small and self-contained part of the whole site) should have only one disposal system. All arisings were treated as likely to be active, even sewage, a special "hot" sewage works being provided to serve this area.

Delay and storage tanks

All delay and storage tanks at Harwell of more than 1,000-gal capacity were of concrete construction and generally below ground level. Construction of the system was relatively expensive and governed in time largely by the weather and available site labour. The Aldermaston wastes were to be predominantly of the alpha type; hence neither underground construction nor massive shielding walls were necessary for protective purposes. Accordingly, to cope with the above difficulties, and to take advantage of workshop capacity, it was decided to use metal tanks throughout. A maximum tank size of 30 ft $\times$ 9 ft 0 in. dia. was chosen since it could be readily transported.

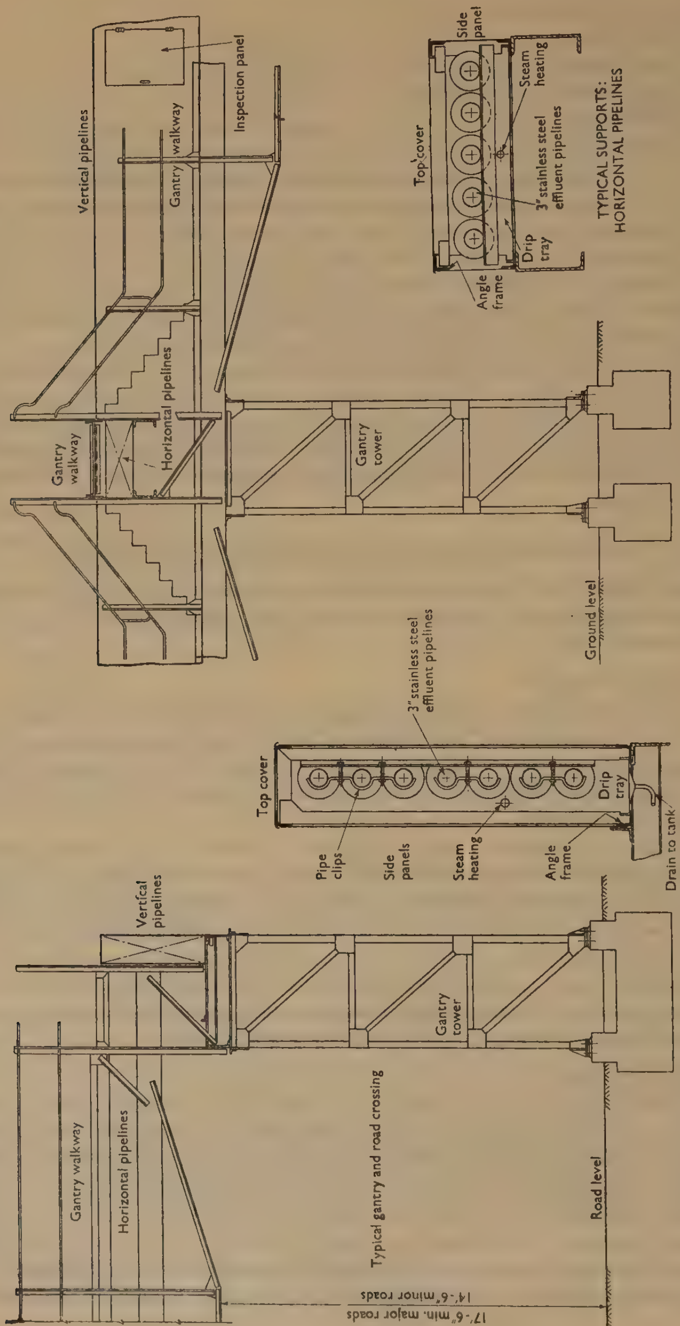


FIG. 1.—OVERHEAD EFFLUENT PIPELINES: TYPICAL BOXES AND SUPPORTS

Effluent mains

Doubts were expressed in the Paper on Harwell[2] on the efficacy of the collection system, formed as it was from stoneware pipe. It was decided therefore to use at Aldermaston welded stainless-steel tube mounted on gantries over continuous drip trays for leakage containment, the pipes being protected from the wind (which would have spread leaks) by side walls and a roof as shown in Fig. 1. The fact that alpha activity alone was being handled permitted such a solution.

Effluent-treatment plant

Less time was allocated for building Aldermaston than seemed reasonable. Much time had been spent at Harwell on the construction of the treatment plant. Mainly for the reasons stated above (in the sub-section on delay and storage tanks) thickeners, clarifiers, filters, etc., were constructed in metal and supported independently of a simple building, the floor of which was made into a saucer to contain effluent leakage and spills.

Discharge pipeline

The Harwell discharge main was sized from limited data to cater for an unpredictable future. The result was a line which was at first too large for the purpose. This, with the need to discharge batches over the longest practicable period, led to velocities of a low order, e.g., about $\frac{1}{2}$ ft/sec. Deposition occurred to a limited degree and occasionally quantities of activity measured at inlet were greater than those measured at outlet. This was made worse by an unexpected development, namely, the tendency for activity to concentrate at exposed metal in the main. At irregular intervals the accumulation was released, and routine examination of inlet and outlet concentrations tended to be irreconcilable, although, in aggregate, conformity was achieved.

A further difficulty originally experienced at Harwell was that of ensuring that the spigot-and-socket joints of the main were leakproof. This was achieved, but at the expense of much effort. An attempt was made to avoid these troubles at Aldermaston by sizing the discharge main accurately, keeping discharge velocities high. To allow for inaccuracies in the assessment of qualities, for extensions, and for possible failures, duplicate mains were laid. Welded joints were chosen as being more likely to be leakproof than spigot-and-socket joints.

Collaboration

With operating department

As was not the case in the early days of Harwell, there was available in 1949 a group of physicists, metallurgists, chemists, and engineers who had acquired experience in the field of atomic energy and who knew, in consequence, a great deal more of the essentials of processes. Accordingly, design teams were formed from these professions to study process problems and produce design studies. These studies resulted in assessments of effluent wastes which were used by the appropriate team in the resolution of its problems.

With industry

It was evident that the immense amount of work that had to be undertaken was beyond the design facilities of the Ministry of Works alone. It was equally clear that many of these problems could best be developed by firms working in allied fields

and so, wherever possible, development contracts were let with appropriate firms. These firms were given basic principles to develop or, alternatively, were associated with current research. The results of this were applied to the design of operational plant. A close collaboration between research, design, executive, constructional, and operational staff thus existed.

With national and local authorities

Reference was made in a previous Paper[3] to the consultation that took place with the Ministry of Health, Port of London Authority, Thames Conservancy, and Metropolitan Water Board to ensure that the radioactive wastes in no way adversely affected the Thames as a source of potable water and an amenity of importance. With this successful experience of collaboration in the maintenance of drinking water tolerance at Harwell, as fixed by the appropriate panel of the Medical Research Council, similar consultation took place for the new works.

Effluent segregation

The principle adopted at Harwell of dividing liquid wastes into two classes—those which in all probability would be active and those which would only possibly be active—was adopted for the new establishment. An additional factor, however, had to be considered because of the probability that citric acid would be used as a decontaminating agent. It was believed at the time that this acid was the most effective of such agents and accordingly arrangements had to be made to handle the arising waste.

It was found that, if citric acid occurs in an effluent which is to be purified, (1) a larger amount of alkali must be added to neutralize it, and (2) it has an adverse effect on known methods of precipitation involving an increase in quantity of chemicals by a factor of 5 to 10.

These effects would give rise to an increase in the amount of active sludge to be disposed of and hence the cost of treatment. To reduce the bulk liquor that would have to be specially handled at the treatment plant, "citric" wastes in laboratories were piped away separately from the other two mentioned, and so each "active" building was provided with three waste systems, namely: (*a*) "probably active"; (*b*) "possibly active"; and (*c*) "citric active." Additional to these systems are: (i) the sewerage system (of which more is said later); (ii) the surface-water system which because of the size of prospective loads was arranged for monitoring at the area outlet; and (iii) local drains for the collection and separation of wastes containing solvents.

Effluent limitation

The principles adopted at Harwell of limiting the body of water that could be linked with activity[2] were followed in the new system, but even more stringently. In addition, cooling-water wastes were restricted in quantity by providing local recirculation systems, the systems being arranged for drainage when the activity level rose. Where possible, water supplies were specifically limited in rate and duration of flow. Particular cases of this description were the "frogmen's"* showers which were arranged for a maximum flow rate of 10 g.p.m. for a duration of 5 min. More-

* "Frogmen" is the term used for staff completely dressed in rubber suits with Perspex helmets, individual air supplies, and inter-communication sets, for the purpose of entering heavily contaminated areas.

over, liquid arising from a rinsing wash was used as a first wash on a subsequent shower user thus ensuring dual water usage and economy.

Tank, pipe, and pump sizing

Effluent quantities

The likely quantity of process waste was assessed from laboratory experiments. It was, in fact, very low and had no material bearing on the total quantities involved. Other wastes (arising from sinks, showers, etc.) were assessed so far as possible on the restrictive devices that were going to be installed in the systems and against the current comparable Harwell usage. Typical figures arising from this exercise for a single building are as follows:

Frogmen's showers (citric)	2,000	gal/month
Frogmen's shower (water)	2,000	,,
Precipitron washing (inlet)	2,000	,,
Precipitron washing (outlet)	2,000	,,
Laboratory washdown	6,000	,,
Laboratory sinks	10,000	,,
Personal showers	7,500	,,
Wash basins	2,500	,,
Fume-cupboard wastes	2,000	,,
Water seal (future)	10,000	,,
Total	46,000	

By such detailed examinations a basic flowsheet was built up, growing as ideas were formulated and crystallized. Fig. 2, Plate 1, shows the flowsheet as it existed after a year's work.

Sump tanks

As already stated, delay tanks were built above ground and gave rise to the need for pumping to them. The tanks for receiving the liquor from individual circuits in the various buildings were known as sump tanks and were made as small as possible to economize in building works. A typical unit is shown in Fig. 3, Plate 1. The size chosen was 250-gal, giving rather more than an hour's capacity on the average flow of the building with the greatest output. Two self-priming pumps of the LaBour type, each of 20-g.p.m. capacity and automatically operated by level control, were connected to them for discharge to the delay tanks. Audible alarms were provided to announce pump failure.

The individual sump tanks were used as water seals to ensure that contaminated air from one room did not pass to another, seals in general not being permitted in rooms because of the hazard they could create.

Delay tanks

At Harwell delay tanks had each been sized for one day's storage. It was felt that this was too limited and that at Aldermaston they could be doubled with advantage. Since the greatest average circuit output was 1,500 g.p.d., 1,500 gal was chosen as the standard tank capacity. All circuits therefore with an output of less than 750 g.p.d. were provided with two 1,500-gal tanks and those with up to 1,500-g.p.d. output with four such tanks. Between the tanks and the preceding pump was placed a 300-gal

splitter box, with the number of outlets therefrom equal in number and connected to the tanks in the system, and arranged so that one of the outlets was always open to a "free" tank, thus permitting the transfer of liquor without the risk of overflow.

Fig. 3, Plate 1, shows the pipe circuits and indicates how the discharge pumps are used for circulation, sampling, and discharge. The pumps discharge to the effluent-treatment plant at a rate of 100 g.p.m., which gives adequate circulation and satisfactory sampling.

The sump, splitter, and delay tanks were fabricated from stainless steel, titanium- or niobium-stabilized to prevent weld decay.

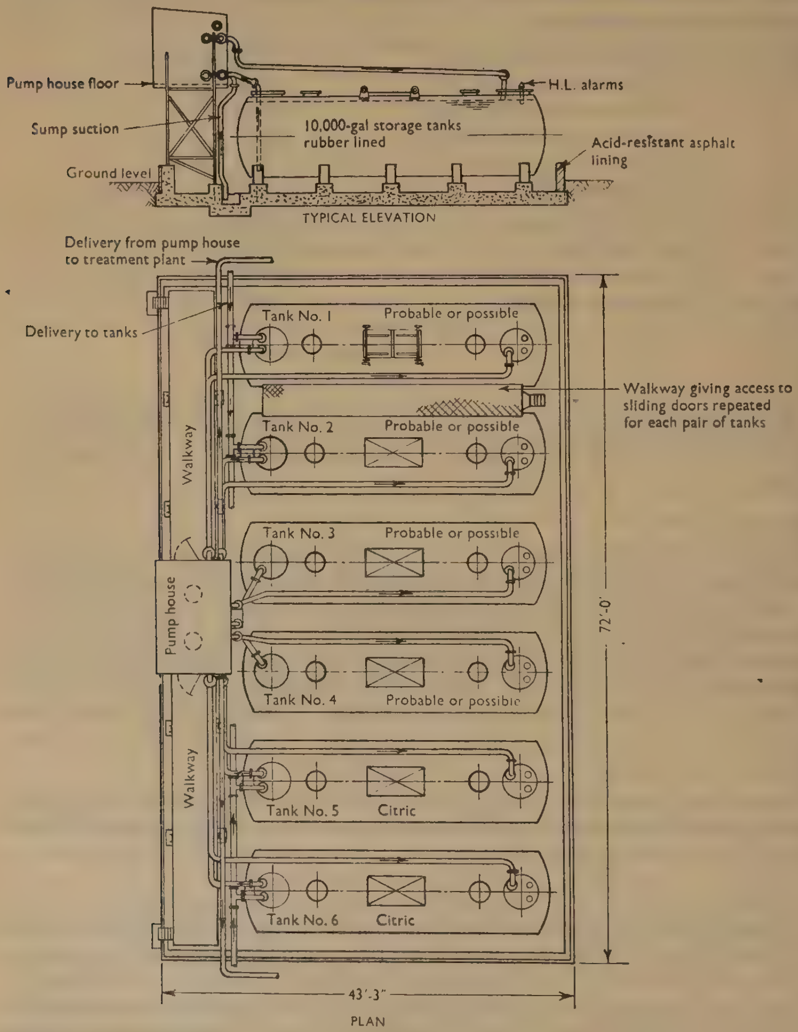


FIG. 4a.—TYPICAL BULK-STORAGE AND PUMP-HOUSE INSTALLATION

Pipelines

Piping was of stainless steel, titanium-stabilized, and was laid from the point of maximum lift with a fall to the effluent-treatment plant so that it would be self-draining. All mains from individual delay-tank systems were kept separate and not connected to common trunk mains, to prevent the possibility of discharge in the wrong direction. The need for the gantry having been established, it was adapted, by the provision of a footway, to serve for the passage of workers between treatment and delay tanks. This permitted the workers concerned (ordinarily regarded as operating in a "hot" area) to pass over clean areas without the necessity for a change of clothing as would otherwise have been the case.

Bulk storage

No effort had been made at Harwell to provide bulk storage of contaminated wastes since the quantities to be handled did not permit of economical provision. However, the following factors had to be considered at Aldermaston:—

(1) Highly contaminated batches of effluent could arise and might need selective treatment.

(2) Plants might be in operation before the treatment and disposal works were completed.

Consequently, since there were primarily two particular contaminants expected, the equivalent of 3 months' storage capacity was made available for each, to the extent of 60,000 gal, arranged as shown in Fig. 4a.

These tanks are fabricated from mild-steel plate and are rubber-lined, the rubber being applied and cured on site. The cost per gallon stored is estimated at 2*s*.

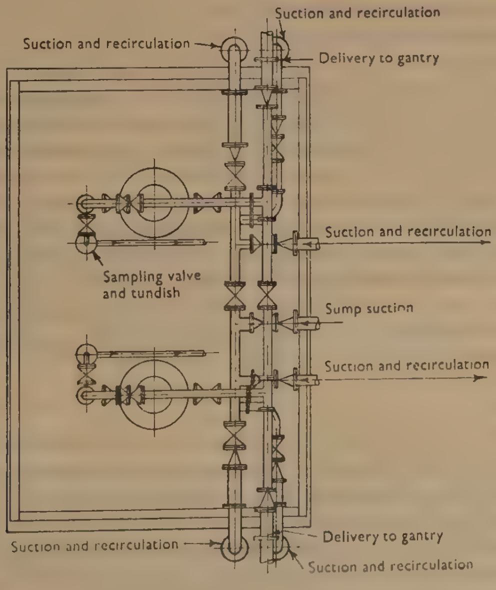


FIG. 4b

Stainless steel was rejected as a material because (*a*) the quantities involved could not be obtained in time (the tanks were a priority item), and (*b*) the need did not justify the cost.

Development of effluent-treatment plant

Scope and requirements of process

The treatment plant was required to remove a number of radioactive materials from solution or suspension in water. Although all the materials were alpha emitters, which eased the problem of design, they each had different chemical properties. Since the radioactivity could never be destroyed the plant could reduce the effluent volume only by increasing its concentration. The concentrated material would ultimately have to be disposed of or stored; it was therefore essential that it should be reduced in volume so far as possible. A consideration of the limits placed on the level of activity of the water discharged into the Thames gave an indication of the order of efficiency required from the plant. It was estimated that the quantity of radioactive material discharged from the laboratories and workshops on the site would not exceed one curie in 24 hours, and this could be accompanied by about 10,000 gal of water. The drinking-water tolerance in general agreement at the time was 1×10^{-11} curies/gal, and the effluent from the site might contain 1×10^{-4} curies/gal. Allowing for a dilution factor of 10^{-4} (the approximate ratio of flow of Thames to site effluent on entry into the Thames) it is apparent that a decrease of radioactivity by a factor of 10^{-3} had to be produced by the plant. These figures are approximate but sufficiently accurate for the purpose of explanation.

It may be of interest to note that the "drinking-water tolerance" is the level of activity in water which a human being might consume for all requirements for the rest of his or her life without ill effect. Also, to bring the figures of curies/gal into perspective it should be noted that the weight equivalent of a curie varies widely from one radioactive element to another but, to quote a typical example, the maximum concentration of an element in the water before treatment might be 20 millionths of a grain per gallon or one three-thousandth of a part per million parts of water, to be reduced by a factor of 10^{-3} in the plant. Even so, each gallon of the material after treatment, assuming that the reduction factor of 10^{-3} was attained, would still be emitting appreciable alpha radiation. Thus, a figure has been obtained for the required performance of the plant, i.e., about 99·9% of all the radioactivity in the water to be removed by the process. An assessment was next made of the likely output of all effluents, of effluents that were likely to be above the tolerance level for discharge (which were designated "probably active effluents"), those which were likely to be below this level ("possibly active effluents"), and thus the total treatment-plant capacity required was obtained.

Type of process

An assessment was next made of the type of process which might be most effectiv in meeting the above requirements. Evaporation is immediately suggested for givin a high reduction factor from feed to product with a low sludge volume, but apart fron the operating costs of such a method there are other factors which are unfavourable The most important of these is that if the liquid is going to be continuously concen trated in an evaporator the concentration of solids (dissolved or suspended) wi increase and the concentration difference to be attained between feed and produc will become not 10^3, but 10^6.

It was not evident from published work that any equipment had been built pre-iously which would meet this sort of specification. Furthermore, it was possible hat some of the radioactive elements, or their compounds present, might have ppreciable vapour pressures. Maintenance might be a severe problem, particularly f heavy scaling or corrosion occurred. Furthermore, the residual sludge for disposal r storage would contain all the dissolved and suspended solids in the water and not ust the radioactive materials. Despite these objections the advantage of having a iece of equipment which could deal with an effluent containing any materials in olution or suspension with reasonable efficiency was such that the design of an vaporator was undertaken. This decision was especially influenced by the fact hat some effluents were likely to arise from "decontamination" processes—i.e., the emoval of radioactivity from surfaces, protective clothing, equipment, etc.—and the olutions required to effect the decontamination were likely to contain complexing gents which might make them difficult to treat by any other means.

Several other processes were considered such as ion exchange and biological treat-nent, but they were discarded for various reasons. The most promising proposal ppeared to be a precipitation process conforming as nearly as possible to standard vater-treatment plant practice, and accordingly most energies were directed towards his end. Certain other factors made this type of process attractive. It could be nade flexible to deal with sudden changes in composition, concentration, and load; could be arranged for complete containment of the liquors and hence be safe; and nown and proven equipment could be obtained in the short time available. More-ver, since such plant had been successful in meeting conventional river regulations lsewhere, i.e., biochemical oxygen demand (B.O.D.) and suspended solid require-nents, it had additional attractions.

From all these considerations it became clear that the problem was to devise, as uickly as possible, a simple coagulation-sedimentation type of process, paying full ttention to the safety of the operators and the general public, and to obtain a product omplying with all the regulations of the various bodies concerned. As a longer-erm project an evaporator was to be developed having a capacity only a fraction of he total effluent output, to deal with any liquid containing material in solution which nade it completely unacceptable to the chemical-treatment plant.

reliminary laboratory investigations

The purpose of the laboratory work was to decide on a coagulant which would be uitable to determine the conditions for that process to give the highest efficiency, nd to obtain information on which to base the design of a pilot plant. The criteria of satisfactory process were established as follows:—

(1) High removal efficiency of radioactivity.
(2) Minimum sludge volume.
(3) High settling rate.
(4) Economy.

he experiments were carried out on a 500-cc scale using a standard technique of nixing, settling, and sampling. Various coagulants were tested of which the only nes giving really promising results were the precipitation of iron and aluminium ydroxides either separately or mixed, phosphate precipitation with lime, or the use f tannic acid with lime. The precipitation of the insoluble hydroxides and phos-hates certainly gave the fastest settling floc and the iron and aluminium hydroxides lso had an efficiency of radioactivity removal of about 95%, but the sludge volume

was appreciable even with the 10-p.p.m. dose that was used. Tannic acid, on the other hand, gave a slightly higher removal efficiency and considerably smaller sludge volume on long standing, but the settling rate was very much slower. The deciding factor between the two was the fact that at the time it was proposed to use citric acid for decontamination purposes and it was found that the presence of citric acid even in quite small quantities reduced the efficiency of the hydroxide floc substantially, whilst the tannic acid was almost unaffected.

Consequently, work was continued on the tannic-acid–lime process although the alumino-ferric method was kept in mind, and in fact, arrangements were made in the final plant to enable it to be used if justified by a change in conditions.

The laboratory effort was next concentrated on determining the optimum conditions for the process. A choice had to be made from a variety of different grades of tannic acid, varying in price from 1*s* 5*d* to 18*s* 9*d* per lb. By a closely controlled series of tests, a suitable grade of tannic acid was found costing about 4*s* 3*d* per lb.

The quantity of tannic acid required for effective treatment, removal efficiency having been balanced against sludge volume, was about 50 p.p.m. The highest removal was at pH 8·5, which was attained by the addition of a dilute lime slurry. The use of a chemical costing 4*s* 3*d* per lb. at a rate of 50 p.p.m. gives a charge of 25*d* per 1,000 gal for tannic acid alone, which must seem considerable, compared with normal water-treatment practice, but when the estimated cost of sludge disposal is 30*s*/gal it can be seen that on a plant treating 10,000 gal of effluent a day the cost of chemicals will be about 21*s*, and of sludge disposal, even if 0·1% sludge volume could be attained, about £15, so that a high price can be afforded for treatment chemicals which give an appreciable reduction in sludge volume.

The advantages of a continuous process in flexibility and economy of operation were apparent from the start, and a laboratory trial was carried out on the continuous injection of chemicals into a stream of liquid, the resultant mixture being collected in beakers and allowed to settle before analysis. There seemed to be no disadvantages involved in this and so the next step was to continue the development work on a pilot-plant scale. At this stage of the work laboratory-scale efficiencies of about 93% were being obtained, so that three consecutive treatments would be necessary to attain the design figure of 99·9% removal. On the pilot plant it was essential that every effort should be made to improve the overall efficiency of the process.

Pilot-plant development

From available water-treatment sedimentation apparatus, it was decided that a continuous sedimentation vessel would best suit the purpose, since it should give minimum sludge volume as well as acting as a sedimentation tank. Accordingly, a pilot-plant model was purchased, 20 in. dia. × 9 in. deep, with a total hold-up of about 10 gal. A 120-gal tank was used for raw effluent which was fed through a pipe into which the chemicals could be introduced *via* a small centrifugal pump and a rotameter. A small mixing pot was introduced at the end of the chemical injection pipe with an electric stirrer, followed by a similar vessel to take the electrodes of a pH meter, and from this the mixture was passed into the central collar of the thickener.

On first running the plant it was apparent that all was not well. Large quantities of calcium tannate floc appeared in the overflow from the thickener, even when the feed rate was reduced to as little as 1 gal/hour, and it was obvious that the calcium tannate particles were too small to have a sedimentation velocity sufficient to obtain a reasonable flow-rate. Accordingly, a flocculation stage was considered for the

purpose of increasing the particle size of the floc and some batch tests were carried out on 3-litre samples from the pH-pot overflow. These were stirred with an arrangement of ten vertical, evenly spaced, ⅜-in. rods rotating at 20 r.p.m. There was no doubt that the floc particle diameter increased very rapidly in the first 30 min after stirring commenced. Samples were withdrawn and analysed at 15-min intervals for 4½ hours after stirring commenced, and the results showed that the efficiency of the process also increased, reaching a maximum after 3 hours. This was confirmed in repeated trials and so a flocculation stage was introduced into the pilot plant, consisting of a cylindrical flat-bottomed vessel with a top feed and side overflow take-off, with an arrangement of fixed and rotating vertical rods acting as stirrers.

When this was incorporated in the plant it was easy to see that a very much improved floc was being produced in the flocculator, which settled rapidly when the stirrer was switched off. The output from the thickener, however, was hardly any better than it had been previously. The cause of this was that the large floc being formed in the flocculator was being broken up by the shear forces acting on it in its passage through the launder carrying it into the thickener, and particularly in leaving the end of the launder. This great sensitivity of the floc was a feature of the process which had to be kept in mind throughout the design. The difficulty was overcome by using a siphon pipe, which ran full bore with both ends submerged, of a convenient diameter to give low turbulence but sufficient to prevent settling-out in the pipe.

Another improvement was introduced after tests had been carried out on a small-scale sand filter which showed itself very effective in removing the last traces of very fine particles of floc which came over the weir of the sedimentation vessel. There were many other minor modifications which cannot be detailed here, but at the end of 6 months the overall efficiency of the plant was established at 95–96%, with a flocculation time of 3 hours, an upflow rate of ⅓ ft/hour in the thickener, and a sand filter as the final stage. With these basic facts and the experience gained in operating the pilot plant, it was possible to proceed with the design of the half-scale plant using the tannic-acid–lime process.

Half-scale plant

The half-scale plant was designed to treat 100 gal/hour of effluent, using a 400-gal flocculator, a 6-ft-dia. thickener, and a 2-ft-6-in.-dia. pressure sand filter. Raw effluent storage of about 6,000 gal was provided, so that long runs could be made with material of constant composition.

Several points arose from the operation of the plant within a very short time. The first of these was the design of the flocculator stirrer which had been based on the experience of the pilot plant and was a large single-cage type rotating at 4 r.p.m. Although the speed in r.p.m. was very low the peripheral speed was about 0·8 ft/sec, compared with 0·25 ft/sec on the pilot plant, and this was found to be sufficient to break up the fragile calcium-tannate floc already formed. At the same time the speed towards the axis was so low that floc was settling out quite rapidly and accumulating in the bottom of the tank. It was apparent that a stirrer of this type could not be upscaled beyond a certain point, which had already been passed.

Finally it was decided to use a propeller-type stirrer consisting of evenly spaced shafts, with approximately one shaft to each 2·5 sq. ft of tank surface and carrying one propeller for each 15 in. of depth, the ratio of propeller diameter to tank diameter being approximately 1 : 8. All the shafts had a common chain drive and after experiment it was decided that a shaft speed of 12–15 r.p.m. was the optimum speed, since above this the floc was small and at lower speeds it was not maintained in suspension.

Propeller design was important also, and it was essential to cut turbulence over the trailing edge to a minimum, so that the propeller imparted only a simple rolling motion to the liquid. Alternate shafts rotating in opposite directions were found to give the most satisfactory result.

Apart from this and other developments on the flocculator, the whole plant was checked item by item and it was then possible to prepare specifications for the full-scale treatment plant. After the various modifications, it operated at a steady removal efficiency of 97% or better, so there was confidence that a two-stage plant should give the required efficiency of 99·9%. Sludge production from the thickener ranged between 0·25 and 0·3% of volume treated, with a solids content of 7–25% according to running conditions, the latter figure being too high for satisfactory flow properties.

This half-scale plant, which had been erected and operated elsewhere, was then re-erected at Aldermaston with a second treatment stage added to treat effluent from the first buildings as they came into service. It was found that although the first stage was very satisfactory, giving a steady efficiency of 97%, the second stage was not quite so good, giving only 94–95% probably because of the reduced concentration of active material initially present in it, so that an overall figure of 99·8% removal was obtained.

The final full-scale treatment plant was developed on these lines, the efficiency in practice, treating high-activity effluent, being maintained at a figure better than 99·9% for a two-stage plant. The plant has given no mechanical trouble requiring servicing of parts made radioactive by contact with effluent, and it has proved to be very versatile in that it has treated a variety of laundry and decontaminant wastes of low activity but high B.O.D. and turbidity.

Demineralizing plant

It has been found experimentally that accurate measurement of low orders of alpha activity existing in potable water is impeded by its natural content of suspended and dissolved solids, and in such circumstances reasonable measurement can be achieved only by evaporating the sample. Such procedure on an effluent system demanding close and frequent quality control would obviously be an embarrassment, and as a consequence it was decided to use a water as free as possible from the objectionable matter. Consequently, demineralized water was adopted for process use in active areas.

The process operates by ion exchange and makes use of columns of granular cation- and anion-exchange resins. The raw water passes first through the cation-exchange column, the calcium, magnesium, and sodium ions thereby being replaced by hydrogen ions. The salts in the water are thus converted to the equivalent mineral acids.

The water next passes through the anion-exchange column and the mineral acids —with the exception of the carbon dioxide formed from the bicarbonates originally present, and a small amount of silica—are absorbed into the resin. The demineralized water is then passed through a de-gassing tower where the CO_2 is liberated by aeration, and finally a small amount of caustic soda is added to raise the pH value.

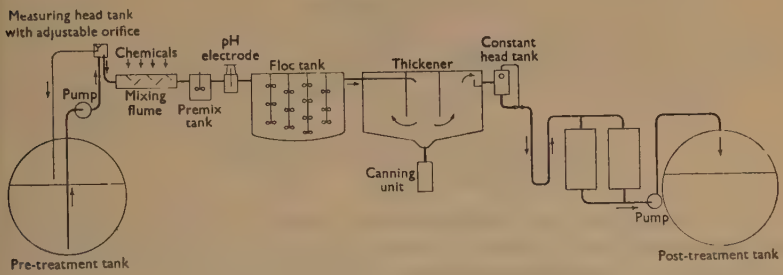
The water thus produced has residual solids of about 20 p.p.m. and a pH of 7·5 it is comparable in quality with distilled water, but is produced in a simpler and cheaper way than by distillation.

Regeneration is necessary at regular intervals when the ion-exchange resins have

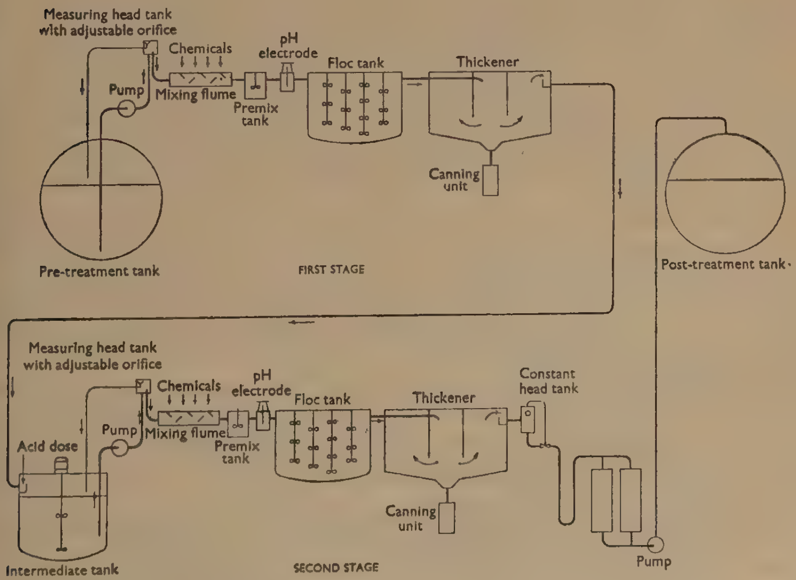
been exhausted. The cation-exchange columns are regenerated with weak sulphuric acid and the anion-exchange columns with soda ash; in each case, the injection of the regenerant is preceded by a short upward wash with raw water to re-grade the ion-exchange material, and is followed by a downward rinse.

Effluent-treatment plant

The effluent-treatment plant is divided into eight sections, each consisting of a complete self-contained unit. These sections are arranged so that they can be operated together in parallel; alternatively, any one section or combination of sections can be worked while the others are shut down. The sections are all basically



(a)



(b)

Fig. 5.—Flow diagrams: (a) single-stage unit, (b) double-stage unit

similar, but three incorporate two stages of flocculation and thickening. These double-stage units were provided to enable the efficiency-of-purification target of 99·9% to be achieved. The flow diagram for a single-stage unit is shown in Fig. 5a and a double-stage unit in Fig. 5b.

The plant is housed in a steel-framed building having brick and Trafford tiling walls and a flat aluminium sheet roof. All major components, with the exception of filters and canning units, are mounted 11 ft 0 in. above ground level on a steel gantry, with operating platforms. The building is divided into three bays, the disposition of the eight sections of the plant in these bays being as follows:—

A Bay
Two single-stage units
One double-stage unit

B Bay
One single-stage unit
One standby double-stage unit

C Bay
Two single-stage units (one as a standby)
One double-stage unit

The untreated effluent from various sources is delivered to storage tanks (known as the pretreatment tanks) situated at ground level outside the building at one end of the three bays. There are several tanks, some with a capacity of 10,000 gal each for holding possible/probable sewage and citric-acid effluents, and two of 1,500 gal each. (See Fig. 12.) Each tank has its own duplicate set of pumps for delivering the effluent to the treatment plant. These pumps are of the LaBour vertical self-priming type of stainless steel with a capacity of 1,200 gal/hour restricted down to 360 gal/hour against 40-ft head. In addition, each large pretreatment tank has a circulating pump for mixing the contents of the tank after acid has been added for pH adjustment. These circulating pumps are also of LaBour vertical self-priming type of stainless steel, with a capacity of 6,000 gal/hour against 30-ft head. This capacity is sufficient for turning over the entire contents of the tank in less than 2 hours.

From the pretreatment tank the effluent is pumped to the measuring-head tank. This is a device for controlling the flow of effluent at a steady constant rate. It comprises essentially a cylindrical tank with an outlet at the bottom, and an overflow, the height of which can be adjusted. The outlet is provided with a fixed orifice, and the height of the overflow is adjusted by means of a capstan. The scale on the capstan is graduated from 100 up to 250 gal/hour.

The effluent enters the measuring head tank at a flow exceeding that required for treatment and the surplus spills into the overflow and gravitates back to the appropriate pretreatment tank. A constant liquid level is thus maintained, as determined by the present height of the overflow, and a constant flow passes through the orifice in the outlet pipe.

From the measuring-head tank the effluent gravitates into the mixing flume, where it receives measured quantities of treatment chemicals. This flume consists of a narrow tank with a sloping bottom and stepped weirs causing turbulence, which results in thorough mixing of effluent and chemicals.

These chemicals forming clear solutions, e.g., tannic acid, are administered by orifice-type dosers mounted on a panel above the mixing flume. The chemical

solutions are prepared in stock tanks, and are pumped to head tanks which feed the dosers by gravity. These head tanks have overflows through which surplus solution returns to the stock tanks. Fig. 6 shows their arrangement.

Each doser comprises a small cylindrical container with an overflow to maintain a constant head, and an outlet pipe having a fixed discharge orifice. The outlet pipe is adjustable in height so that the head over the orifice, and consequently the rate of discharge, can be altered. A needle valve on the inlet is provided to control the inflow of solution to the container at a rate slightly in excess of the predetermined rate of dosing. The surplus chemical gravitates back to the stock tank *via* the overflow.

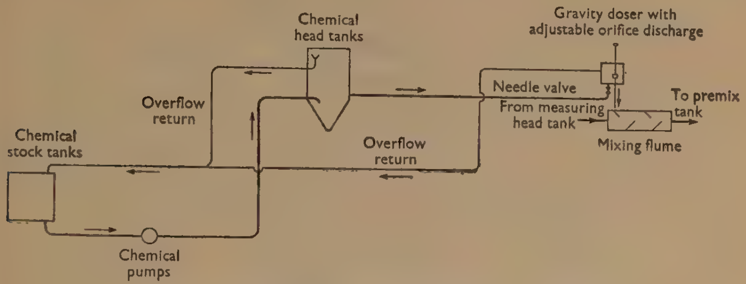


FIG. 6.—ARRANGEMENT OF CHEMICAL DOSING GEAR

The rate of dosing is set by means of a pointer linked with the orifice tube. The pointer is connected with a motorized geared unit whereby the orifice can be raised above solution level by remote control to stop dosing. The orifice can similarly be returned to its original setting when dosing is re-started.

Lime slurry is delivered into the mixing flume by a motor-driven variable-stroke measuring and injection pump located immediately below the flume. The pump draws lime emulsion from a cylindrical stock tank fitted with a motor-driven propeller-type agitator; the motor driving the pump is under automatic control from a float switch arranged so that lime dosing takes place only while effluent is flowing through the mixing flume.

The amount of slurry delivered by the lime pump is adjustable while the pump is running. The pH value of the effluent passing through the flume, as shown by the pH indicator on the dosing panel, can be adjusted immediately.

After leaving the mixing flume, the chemically treated effluent passes into the premix tank, which is a small stainless-steel vessel provided with a propeller-type agitator driven from a small d.c. motor. This motor is controllable to give any required speed between 100 and 1,000 r.p.m.

On the outlet from the premix tank is a pH-measuring head which was a special development. It consists of an electrode assembly fitted into a rubber-lined cast-iron body; a by-pass is provided so that the entire electrode can be withdrawn for cleaning and servicing while the plant is in operation. The electrode element is coupled with the indicating and recording instrument on the chemical-dosing panel. Fig. 7 shows the general design.

From the pH-measuring head the effluent passes to the floc tank which consists of a cylindrical stainless-steel dished-bottom tank having dimensions of 6 ft 0 in. dia. × 5 ft 0 in. high, holding 750 gal. The tank has a double-seated valve

arrangement, operated by hand from a capstan, so that the flow through the tank can be in an upward or downward direction as required.

The stirring mechanism in the tank consists of twelve vertical shafts each fitted with 10-in.-dia. propeller-type impellers. The shafts are driven through a chain and pulleys by a 1-h.p. motor through a four-speed reduction gearing giving a choice of 19, 12, 9·5, and 6·1 r.p.m. The tank is also fitted with a high-speed agitator for lifting any floc which has settled and consolidated in the bottom dishing. Fig. 8 shows the main features of the unit.

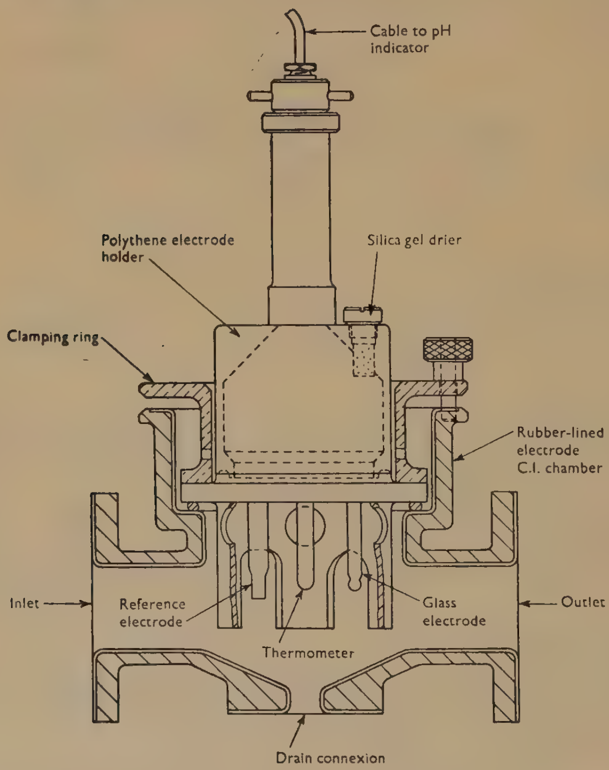


FIG. 7.—SECTION THROUGH pH ELECTRODE

From the floc tank the partly treated effluent flows to the thickener. This consists of a cylindrical mild-steel tank of 10 ft 0 in. dia. × 6 ft 6 in. deep at the centre, lined internally with sheet rubber, and having a capacity of approximately 2,500 gal. The bottom of the tank slopes towards the centre which forms a collecting well, and scraper gear is provided so that the sludge is slowly and continuously swept into this well. The scraper blades are attached to a double arm fixed to a vertical central shaft which is driven through a gear box and worm gearing from a $\frac{3}{4}$-h.p. motor. The variable gear gives a speed range of 24 to 8 rev./hour.

The effluent enters the thickener from the floc tank through a central downtake

tube; it then rises slowly and is withdrawn through a number of orifices provided in the annular collecting-trough at the top of the tank.

A constant-level tank is provided on the outlet from the floc tank through which the effluent passes on its way to the pressure filters. This level control destroys surplus head and thereby ensures a constant head on the filters.

There are two filter units operating in parallel for each section of the treatment plant. Each unit consists of a closed vertical rubber-lined mild-steel cylinder of 2 ft 6 in. dia., containing a layer of filter sand supported on graded layers of pebbles. Embedded in the bottom layer of pebbles is a collecting system through which the filtrate is withdrawn, and externally the filter has a frontal piping assembly and hand-

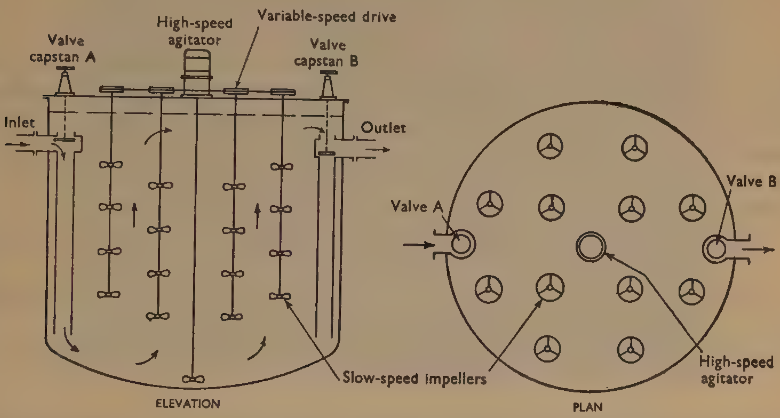


FIG. 8.—PRINCIPLES OF FLOC-TANK. THE DIRECTION OF FLOW MAY BE REVERSED BY LOWERING VALVE "A" AND RAISING VALVE "B"

operated control valves. There is also a manually rotated rake used for agitating the sand bed during the cleaning process. The teeth of the rake project into the sand; they are attached to a cross-bar secured to a vertical shaft which projects through a gland in the lid of the filter. The rake is rotated by a large hand-wheel attached to the shaft.

The filters are provided for removing all traces of suspended matter which remains in the effluent after flocculation and sedimentation. From the constant-head tank the effluent flows to the inlet point of the filters and thence downwards through the sand beds. The suspended matter is trapped in the upper part of the bed, and the filtered effluent is drawn off through the collecting system and thence flows to the post-treatment storage tanks.

Filter cleaning is necessary when the head lost through the filters—indicated by gauges fitted on the inlet and outlet pipes—reaches a predetermined figure. The operation is carried out by passing a reverse flow through the filter while the rake is rotated; this loosens and carries away the flocculent matter trapped in the sand.

The water used in cleaning is drawn from one of the raw-water tanks on the roof of the plant house, and the waste water from cleaning is returned to the appropriate pretreatment storage tank for retreatment.

In addition to this cleaning, arrangements are incorporated whereby the filters can be flushed with sulphuric acid. For this purpose, dilute acid is pumped from the

storage tank in Bay A chemical store. After passing through the filters, the acid is delivered into the wash return tank above Bay C and thence to the appropriate pre-treatment tank. After acid flushing, the filters are rinsed with water.

In each bay, at the pretreatment end of the building, a cabinet is provided for housing the control equipment for each section of the plant. Panels in front of these cabinets carry pH indicators and recorders, level indicators for the pretreatment and post-treatment tanks and for the various chemical tanks, signal lamps for all the motors driving pumps, agitators, etc., and push buttons for the various circuits. The master panel is situated in Bay B, from where the operation of any section of the plant may be seen.

Canning units

Because the sludge produced would be highly toxic, its handling and disposal presented considerable problems. Various methods were considered and enquiries were made of Government Departments to determine whether advantage could be taken of existing experience. Unfortunately, these enquiries disclosed that there was no available information likely to be of any great help, and it was thus necessary to develop a new technique. The basic requirements were:—

(1) The sludge was to be discharged into cans which would be sealed and dumped into the sea.
(2) The cans were to be strong enough to withstand even abnormally rough treatment without leaking.
(3) Operators were to be completely protected during the entire operation of filling and sealing the cans.

The size of the canning equipment was dependent on the volume of cans to be used, and it was eventually decided that 5 gal was the largest measure which could be handled conveniently.

A prototype can of this size was accordingly designed and constructed from $\frac{3}{16}$-in. mild-steel wrapper plate and $\frac{1}{4}$-in. ends. The neck-piece was provided with a protecting shroud and was arranged to be closed with a rubber bung and externally screwed-on steel cap. As a final safety precaution to prevent accidental opening after filling, a hole was provided in the top of the cap to receive a hardened-steel pin. When rammed home, this pin destroyed the thread and made subsequent opening impossible.

This prototype was tested hydraulically to 100 lb/sq. in. and was then dropped from a height of 20 ft on to spiked angle-irons. Only slight distortion of the neck shroud resulted, the body and cap remaining sound and undamaged. The design was therefore accepted as satisfactory.

The equipment eventually adopted for filling the cans is shown in Fig. 9. A closed stainless-steel cylinder holding 5 gal surmounts a rectangular cabinet made from sheet steel and sheet Perspex attached to an angle-iron framework. The measuring cylinder is provided with Perspex windows and has an outlet with two connexions at the bottom to which are attached the valves for controlling sludge entry and exit. The valves are of glandless diaphragm pattern operated by a removable key inserted through orifices cut in the Perspex side of the cabinet.

Beneath the can-filling valve there is a downward-pointing nozzle arranged to fit snugly into the neck of the can; an air line is connected to the nozzle so that immediately after a can has been filled the nozzle is blown through to remove any drops of

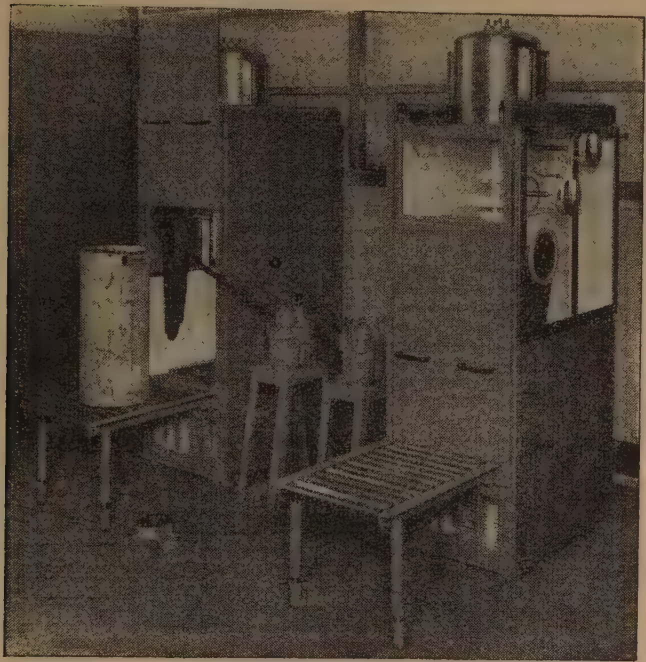


Fig. 9.—Canning unit

sludge. The action of closing the can-filling valve automatically opens up the air supply.

The door of the cabinet is of guillotine type supported by counterweights, and is arranged so that it cannot be fully closed until the rubber bung from a can has been inserted into the lock. This eliminates the risk of a can being put into position under the filling nozzle with the bung still in place in the neck-piece.

To fill the can the main door of the cabinet is raised and the can, with cap and rubber bung removed, is pushed on the roller conveyor into position on the self-centering table under the nozzle. The rubber bung is then placed in the door lock and the door is closed.

A hydraulic jack is provided to raise the can a few inches so that the nozzle penetrates into the neck-piece; as the can rises, the neck shroud engages a lever which removes a shutter from the orifice giving access to the spindle of the can-filling valve.

The key is then applied to the measuring-cylinder inlet valve, and this valve is opened to allow sludge to flow into the cylinder. When the latter is filled, the valve is closed; the key cannot be removed from the spindle until closure has been effected.

The operator then transfers the key to the can-filling valve which he opens so that the charge of sludge flows into the can. He closes the valve when the measuring cylinder is empty.

This system of valve operation ensures that both valves cannot be opened simultaneously. It will be seen that the key can be applied to or removed from the spindle of the measuring-cylinder inlet valve only when the valve is shut; and the orifice

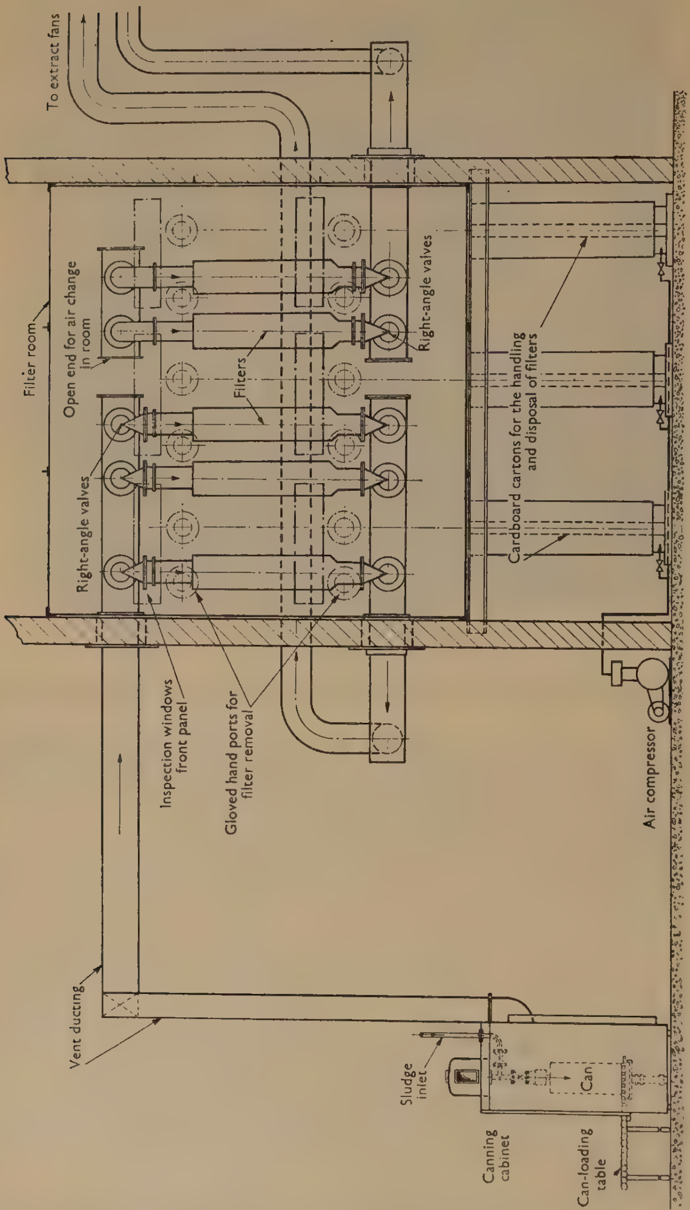


FIG. 10.—SLUDGE CANNING UNIT WITH AIR-FILTER ROOM

iving access to the can-filling valve is closed by the shutter until a can is correctly in osition under the filling nozzle.

The next step is to lower the can by means of the jack. The neck-piece of the can thus withdrawn from the nozzle, the rubber skirt in the neck-piece wiping away any rops. The operator then inserts his arm into a rubber glove attached to an opening the Perspex side of the panel, removes the bung from the door lock, and pushes it nto the neck-piece. The door is then opened and the can withdrawn on the con-eyor. Finally, the steel cap is screwed down and the steel pin is driven home.

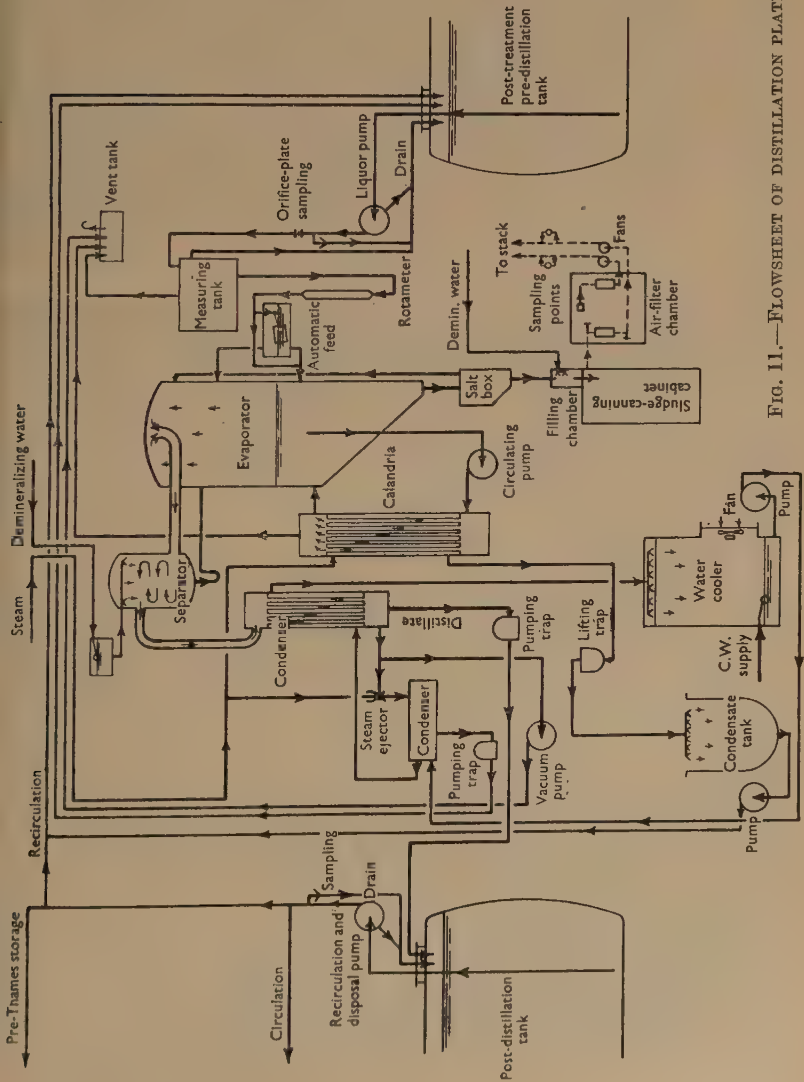


FIG. 11.—FLOWSHEET OF DISTILLATION PLATE

As an added safety precaution, the rubber bung for the cap is so shaped that it will remain in position even if the can is knocked over before the steel cap has been fitted.

There are ten canning units all mounted at ground level below the effluent-treatment plant. Six serve the thickener and four were provided for use with the citric cones. They are connected to the main air-extraction plant where contaminated air is purified. Fig. 10 shows the air-filtration plant.

Distillation plants

Based on the data obtained from the research in the section on the development of the effluent-treatment plant, the plant shown schematically in Fig. 11 was devised.

The evaporators were specially developed to give a low entrainment ratio, better than the 10^6 contamination reduction factor between boil-up liquid and condensate. A type of forced-circulation vacuum evaporator was used with external calandria, the vapour velocity in the evaporator body being kept down to 1 ft/sec with a large vapour space before internal baffles and with an external cyclone separator. In practice it has been necessary to use these evaporators only very occasionally, although their performance has been checked as giving the required 10^6 entrainment ratio. The effluent is again tested in the post-distillation tanks and then passed forward to a series of 10,000-gal pre-Thames collection tanks where it is given a final activity check before discharge to the Thames.

The physical disposition of the units with respect to the pre- and post-distillation tanks is shown in Fig. 12, Plate 2, the pre-distillation tanks being of course the post-treatment tanks of the precipitation processes. Again it should be noted that the plant was built in a house with the floor of saucer formation to retain leakage.

Means of disposal for liquid effluent

Possibilities for means of disposal were:—

(1) By road tanker to the sea.
(2) By barge through the Kennet-Avon canal.
(3) By Pluto pipeline to the sea.
(4) By pipeline either to the River Kennet or to the River Thames.

Research soon showed that the canal was not in a good state of repair and that the cost of making it serviceable would be great. The time required for the operation was estimated to be greater than could be permitted if the overall completion target date was to be achieved. It was also evident that the Pluto pipeline system (which ran quite close to the site) would have to be closely examined and tested before a decision could be made on its use. There was also a possible clash of interest with a national fuel-distribution policy, the unravelling of which could well have taken up precious time and therefore no great attention was paid to the further development of Pluto.

Transport by road tanker to seaboard for eventual disposal by ship was examined. A thorough appreciation of the following hazards arising from spillage during transport was made:—

(1) Temporary contamination of local water supplies.
(2) Contamination of crops.
(3) Contamination of individuals involved in an accident.
(4) The spread of activity by cars or by foot.
(5) The contamination of vehicles and the effects that could arise therefrom.

The worst and most improbable ways of activity giving rise to these effects were hen assumed and it was found that it would be unwise to convey wastes in this ashion if the concentration of activity exceeded a few thousandths of a gramme per allon.

Further calculations, assuming the use of 500- and 1,000-gal tankers, journeys to ondon or Bristol, and including proper allowances for operation, interest, and epreciation, showed a charge varying from $2\frac{1}{4}d$ to $4\frac{1}{2}d$/gal, and therefore this method f handling was rejected as hazardous and expensive. These charges do not include ctual disposal at sea.

The Rivers Kennet and Thames were finally considered for disposal purposes and hough the Kennet was attractive owing to its closer proximity to the site, the 'hames was chosen because (*a*) discharge into the Kennet would eventually reach the 'hames, and (*b*) the rate of flow at the disposal point would be greater in the latter han in the former.

After negotiation, work proceeded on the basis that the extreme dry-weather flow f the Thames at the point of discharge was 100,000,000 g.p.d., of which 50,000,000 al was assumed to have been contaminated to the acceptable limit by the Harwell ischarge upstream.

The effluent-discharge system

're-Thames bulk storage

The calculated discharge rate was 25,000 g.p.d. It was assumed that three mes this quantity should be held on the site before delivery to the Thames to permit n arrangement of 1 day being spent on the final checking of the quality of the liquor, day's supply being received, and 1 day's supply being released. Inasmuch as),000 gal had been determined as the standard storage tank size, nine 10,000-gal ild-steel tanks were provided, all being painted internally with a bituminous enamel, is being accepted as a reasonable protection against corrosion by the treated liquor.

ipelines to the Thames

There was an available head of about 220 ft between the site and the River Thames d it was decided that this should be taken advantage of for discharge purposes in e event of a power failure. As has been stated elsewhere[2] the preference has been discharge effluents by pumps to avoid the troubles which can arise by accidental scharge from a gravity system. Hence, whilst the mains were designed to suit the ailable gravity head, this method of discharge was regarded as an emergency ternative.

The topography of the route made it necessary for the main to be comprised of pes of 5 in. and 3 in. dia., the first 5–6 miles being laid to a fall of about 1 in 1,500. e velocities achieved were approximately 1 and 2·7 ft/sec; this was a considerable crease on the Harwell practice and overcame the difficulty mentioned on p. 627. e pipe-sizing calculations carried no margin for inaccuracy, and so two similar ains were provided. To take full advantage of this duplication, valved "cross-ers" were provided at approximately $\frac{1}{2}$-mile intervals, thus ensuring almost the full pacity of both mains in the event of a single break, and the full capacity of one ain in the event of two breaks, provided that the two breaks did not occur in both ains in identical sections. The valves used were of the glandless diaphragm ttern to prevent the enlodgement of activity.

The valve pits for the "cross-overs," air-release valves, etc., were lined with

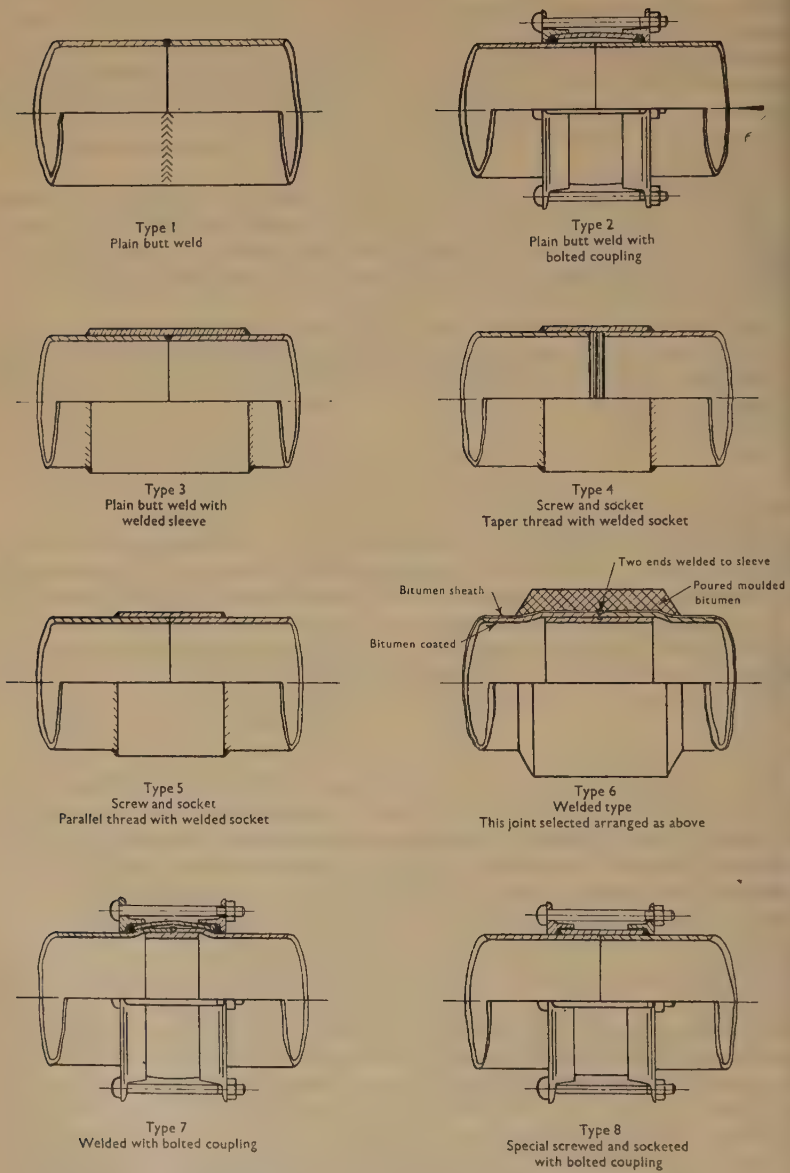


Fig. 13.—Joints examined for effluent mains to the Thames

-in.-thick steel plate, bitumen-painted and welded to the mains to ensure that eakage was retained within the pits. Where mains run under roads and streams they were not taken over the latter for obvious reasons) they were housed in steel ucts terminating in steel-lined manholes in which leakage could be detected and etained. If the precise tolerance figures were to be achieved in the Thames, the oncentrations of activity in the main could have been some thousands of times the ermissible figure. Leakage retention was therefore an important issue. As a esult it became necessary to locate all the wells in the area and lay the main at the arthest practical distance from them.

Eight joints of different kinds (Fig. 13) were examined and pressure-tested before ne (Type 6) was adopted. This withstood an internal pressure of 700 lb/sq. in. f nitrogen and had the merit of presenting a flush internal surface which reduced e possibility of an internal accumulation of radioactivity.

lains protection

An examination of soil on the route showed that external protection of the main ould be necessary. A survey of the route was then made to determine whether athodic protection might offer a solution to this problem and it was indeed found to e so.

However, it transpired that:—

(1) It would not be cheap since controlled potentials would be required.
(2) A survey of other services would be necessary and bonding thereto would have to be undertaken if they were not to be adversely affected.
(3) Negotiation for, and provision of, the requisite electricity supplies could involve a greater expenditure of time than could be spared.

It was therefore decided that the mains should only be bitumen-sheathed. The urve shown in Fig. 14, produced since the installation was completed, shows potential ifferences which indicate that the main is adequately protected. Where the sheathig was cut to permit welding of joints, bitumen was cast around the open length.

Internal corrosion of the mains had to be catered for and so the dosing gear shown Fig. 15, Plate 2, for adding calgon proportional to the rate of flow, was provided.

ischarge in river

Having decided to effect discharge through divergent nozzles placed across the ed of the river, consultation with Thames Conservancy and local authorities took lace and the location was agreed, it being specified that the nozzles had to be not less an 8 ft 6 in. below minimum summer-flow level. It was further specified that no oove-ground works could be erected at or near the river bank.

It was soon apparent that a flow of 50 g.p.m. could not be readily mixed with the ain body of water since the number of nozzles required to cover the stream would volve small nozzle openings very liable to blockage. Accordingly, and rather bitrarily, it was decided that the gross flow through the nozzles should be 250 g.p.m. ade up of 50 g.p.m. of effluent and 200 g.p.m. of fresh water. On the assumption at about six nozzles would be required, individual submerged-nozzle experiments ok place in a London swimming pool, and then in a diving tank, to determine e required shape, dimensions, and pressure required to give the desired flow and erlap effect. Fig. 16, Plate 2, shows the nozzle details and performance.

It was decided to provide two discharge pipes across the river, each of 4-in. dia., ted with nozzles at 16 ft 0 in. centres. The river was dredged to give the required pth, the dredging uncovering a hard bottom which was exposed for some distance

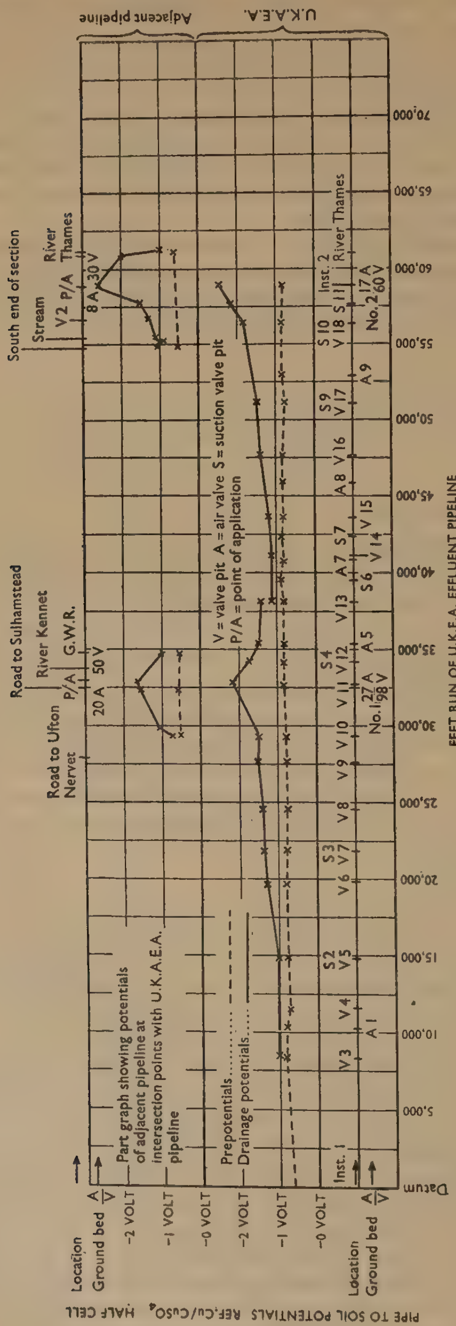


FIG. 14.—PROPOSED CATHODIC PROTECTION OF EFFLUENT PIPELINE
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pstream to delay silt build-up near the mains. The mains were then held in position s shown in Fig. 16, Plate 2.

dditional river works

Having decided that additional water was required for augmenting the effluent ow, a search was made for the requisite 200 g.p.m. of clean water. The location of ie river control house having been fixed at the sewage works, as shown in Fig. 15, late 2, a jack well was sunk which unfortunately gave only 100–150 g.p.m. at a epth of 18 ft 0 in. Permission was granted for 200 g.p.m. to be drawn from a brook, iere being an obligation to pass 200 g.p.m. downstream. Since these two sources ı very dry weather could not provide the necessary quantity, a 5-in. main as laid from the house to the river, so that from there a mobile fire pump could rovide the requisite flow. The water so provided was mixed in a water-to-water ector with the 40–50 g.p.m. of effluent. Larner Johnson surge-control valves were tted in front of the ejectors with opening and closing times of $2\frac{1}{2}$ min and $6\frac{1}{2}$ min spectively, the timing having been calculated from a Paper by Lapworth.[4] A hematic arrangement is shown in Fig. 15, Plate 2.

emote and other electrical controls

Bulk storage and pre-Thames pump-house.—Each of the nine tanks is fitted with three ectrodes, one at a high level, one at a low level and one (which is the maintain-g or feed electrode) at a lower level. These permit audible (for high level) and sual indication of levels and are used also in the automatic (and manual) discharge the tanks.

Control cable.—The site pump-house and river control house are linked over the -mile distance by an eight-core 3/0·036 V.R.I. lead-covered, single-wire armoured id served cable. This cable is laid alongside the pipeline and passes through ghteen valve pits in each of which is a field-telephone jack point. Four cores of e cable are used for control circuits between pump-houses, two cores carry a -V d.c. supply, and the remaining pair are used for telephone communication be-reen pump-houses and/or valve pits.

Starting (from site).—The system is switched to automatic and, by means of a tary selector switch, the operator selects the tank to be cleared. If the tank is ll or contains a liquid above the intermediate electrode, a light on the pipeline ntrol relay is illuminated. The line-control relay is energized and this relay erates the discharge pump. Simultaneously, one circuit of the control cable tween pump-houses is completed, thus operating a relay in the Thames pump-use, which in turn energizes not only a green lamp to indicate that the line is nning, but the raw-water injection pump and the solenoid valves controlling the rner Johnson valve.

Starting (from Thames pump-house).—The above sequence can be initiated from e Thames pump-house, provided that one of the bulk-storage tanks at the pre-ames end has been previously selected for discharge by means of the rotary selector itch at that end.

General note on operation.—The above description is the operation for one line, but t is desired, the two lines can be run simultaneously since the equipment is dupli-ted and the tank-selector switches are not interlocked; similarly, any two of the lk-storage tanks can be discharged together down one line.

If gravity flow is required, the same procedure as that described for the starting

operation is followed, but, in addition, the pump-motor starters at the site end must be switched to the "off" position.

Static and flow tests

Stringent pressure tests were applied to the sections of the effluent mains and then to the composite lines. Storage tanks were likewise tested. The usual electrical tests were applied to all cables and motors, and finally the capacities of the lines in gravity and pressure flow were determined.

These procedures were carried out in great detail by a combined team comprising the contractors' personnel, Ministry of Supply (now United Kingdom Atomic Energy Authority), and Ministry of Works staff.

They disclosed that each line under gravity conditions gave a flow of 40–42 g.p.m. with a maximum surge pressure on closing of 106 lb/sq. in. on a valve-closing time of 6½ min. Under pumping conditions the flow-rate achieved for each line was about 48 g.p.m. with a maximum surge pressure of 160 lb/sq. in., i.e., well inside the capacity of the main.

Testing of discharge system

Testing fell naturally into two parts; first, the ability of the system to handle the requisite quantity of liquid safely; secondly, the ability of the system to give the required diffusion in the Thames.

The results relating to the first part have been briefly stated, the facilities required to produce them being quite orthodox. Determining the effect in the Thames required rather more unusual methods.

It was considered desirable to show to all concerned that the effluent rapidly dispersed in the river and caused no undue disturbance. Consequently, means of exhibiting the effluent flow to the naked eye were sought and conventional dyes were discarded for this purpose because of the perturbation their appearance might cause downstream. Ultimately, fluorescein was used as a dye in concentrations so low that it could not be seen other than when exposed to ultra-violet light.

As a consequence, the arrangement shown in Fig. 16, Plate 2, was devised, the boom walkway being surmounted with discharge lamps directed over the sparge pipes. Motor-boats carrying observers and discharge lamps moved upstream to determine the line at which complete mixing occurred. A diver located the underwater plume effect. Fluorescein was injected in the proportion of 0·01 p.p.m. against a river flow of 200,000,000 g.p.d., and a simulated effluent of 250 g.p.m. was provided by the mobile pump on the river bank. The flow pattern produced is shown in the figure referred to. Surface disturbance was noticeable at a flow-rate of 175 g.p.m. but was in no way serious at 250 g.p.m. The tests, apart from those on surface disturbance, were undertaken at night.

The quantity-head curve derived from these tests showed that a pressure of 55 ft 0 in. at the river bank was required to produce the requisite result, and this was used to determine the form of the water injector mentioned previously.

The dispersion of effluent was considered satisfactory by all concerned.

Construction effort on pipeline and river works

Work commenced on the construction on 15 March, 1951. One line was ready for emergency discharge by mid-September, 1951, one line was fully operational

THE CONTROL, CONVEYANCE, TREATMENT, AND DISPOSAL OF RADIOACTIVE EFFLUENTS FROM THE ATOMIC WEAPONS RESEARCH ESTABLISHMENT, ALDERMASTON

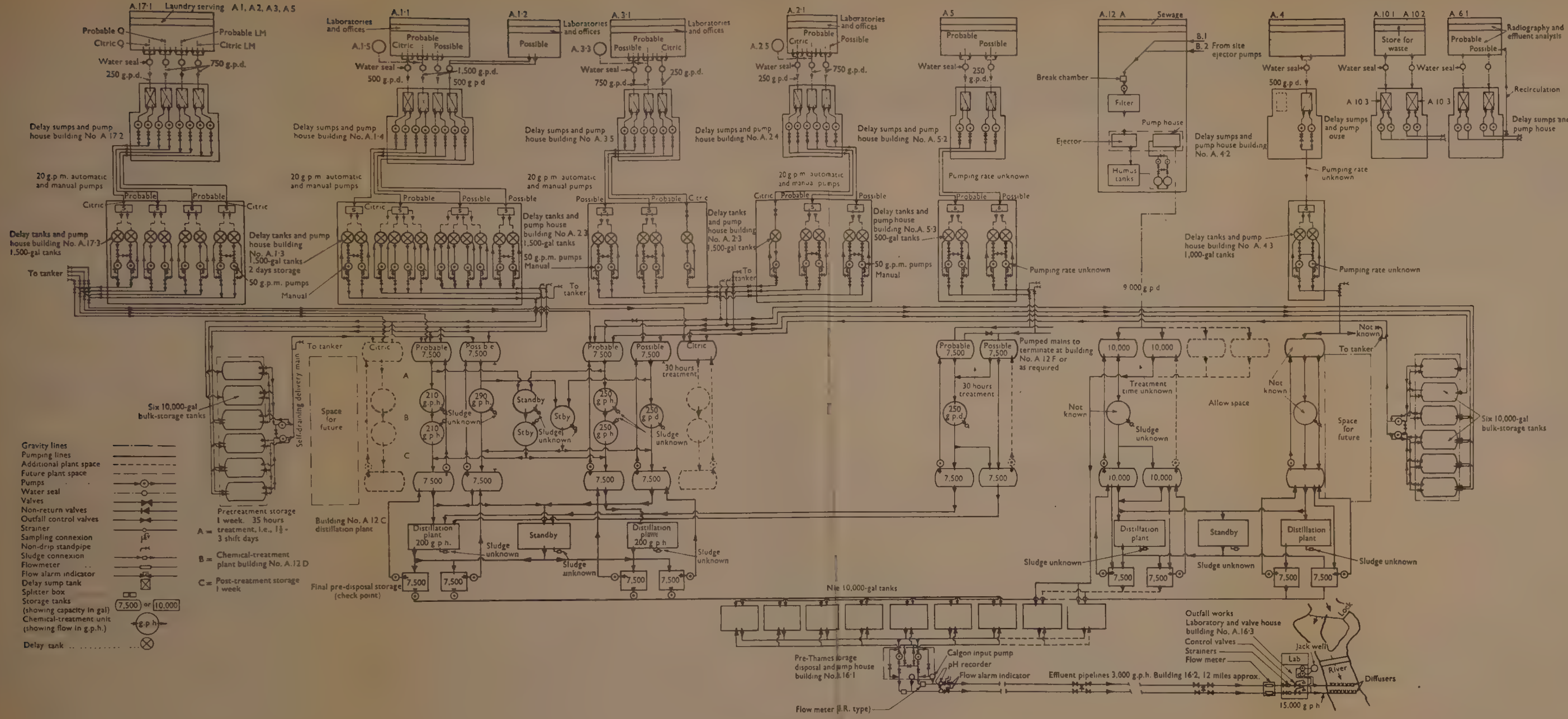

FIG. 2.—APPROXIMATE FLOWSHEET, OCTOBER 1950

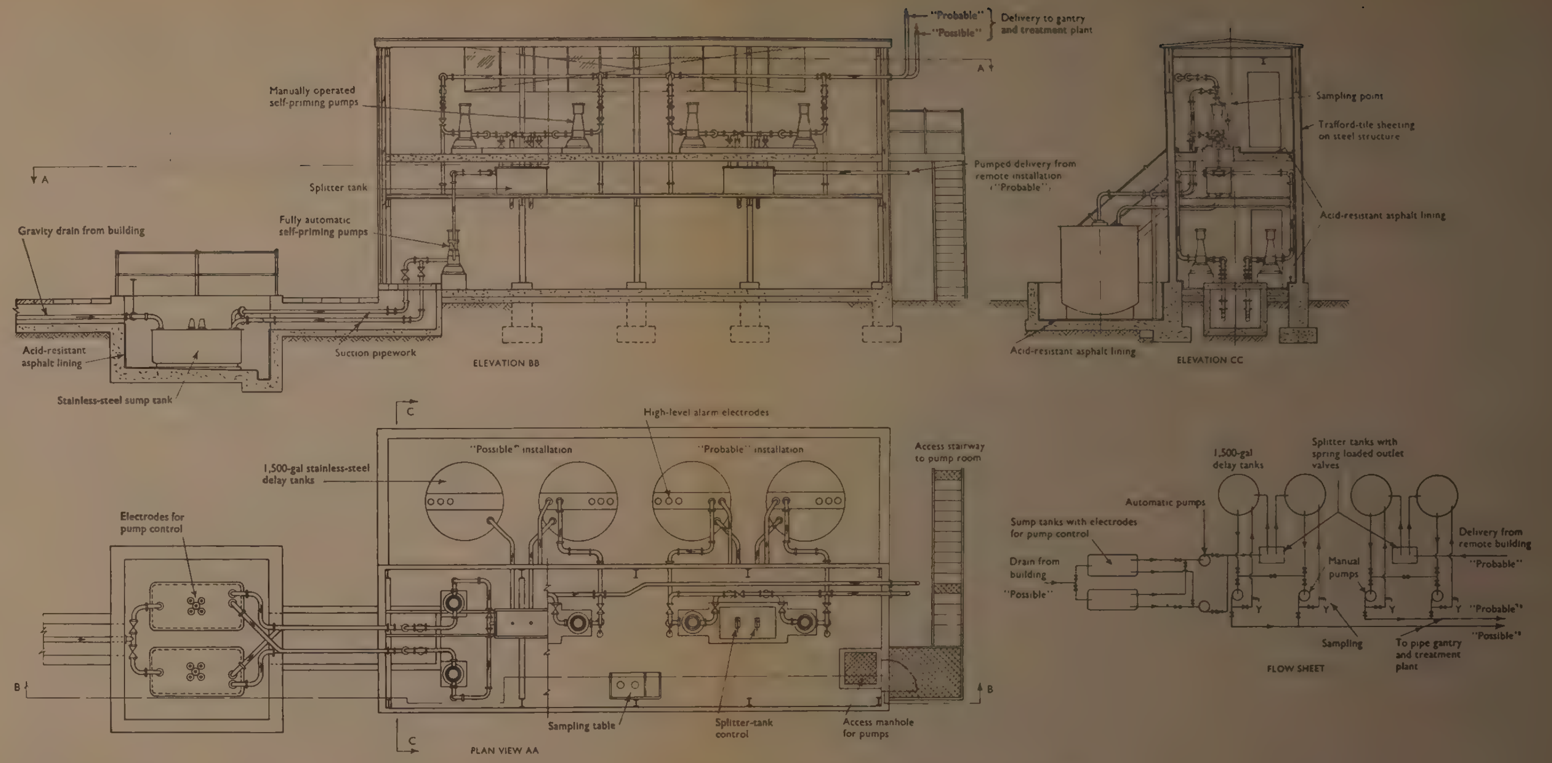

FIG. 3.—TYPICAL SUMP AND DELAY-TANK PUMPING INSTALLATION WITH FLOWSHEET

W. L. WILSON, P. A. F. WHITE, and J. G. MILTON

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THE CONTROL, CONVEYANCE, TREATMENT, AND DISPOSAL OF RADIOACTIVE EFFLUENTS FROM THE ATOMIC WEAPONS RESEARCH ESTABLISHMENT, ALDERMASTON

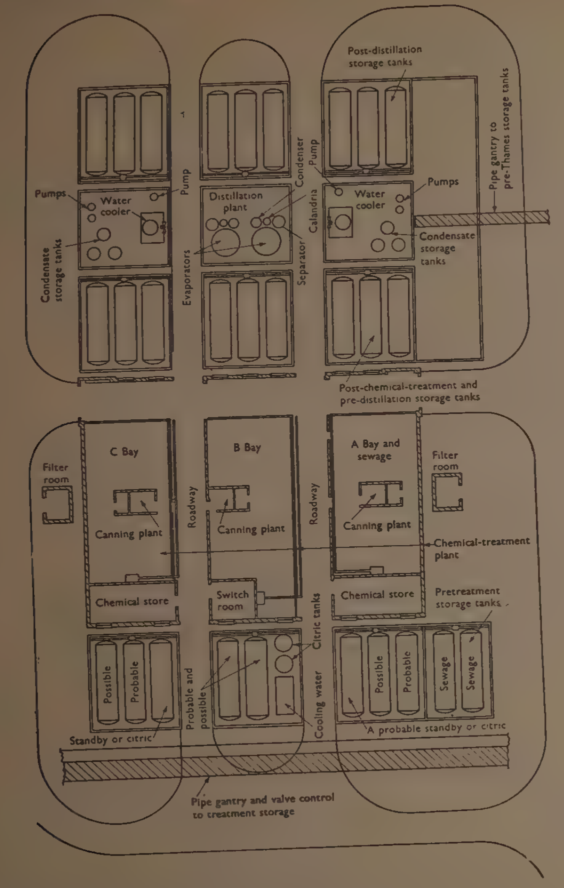

FIG. 12.—TREATMENT AREA: DISPOSITION OF 10,000-GAL TANKS

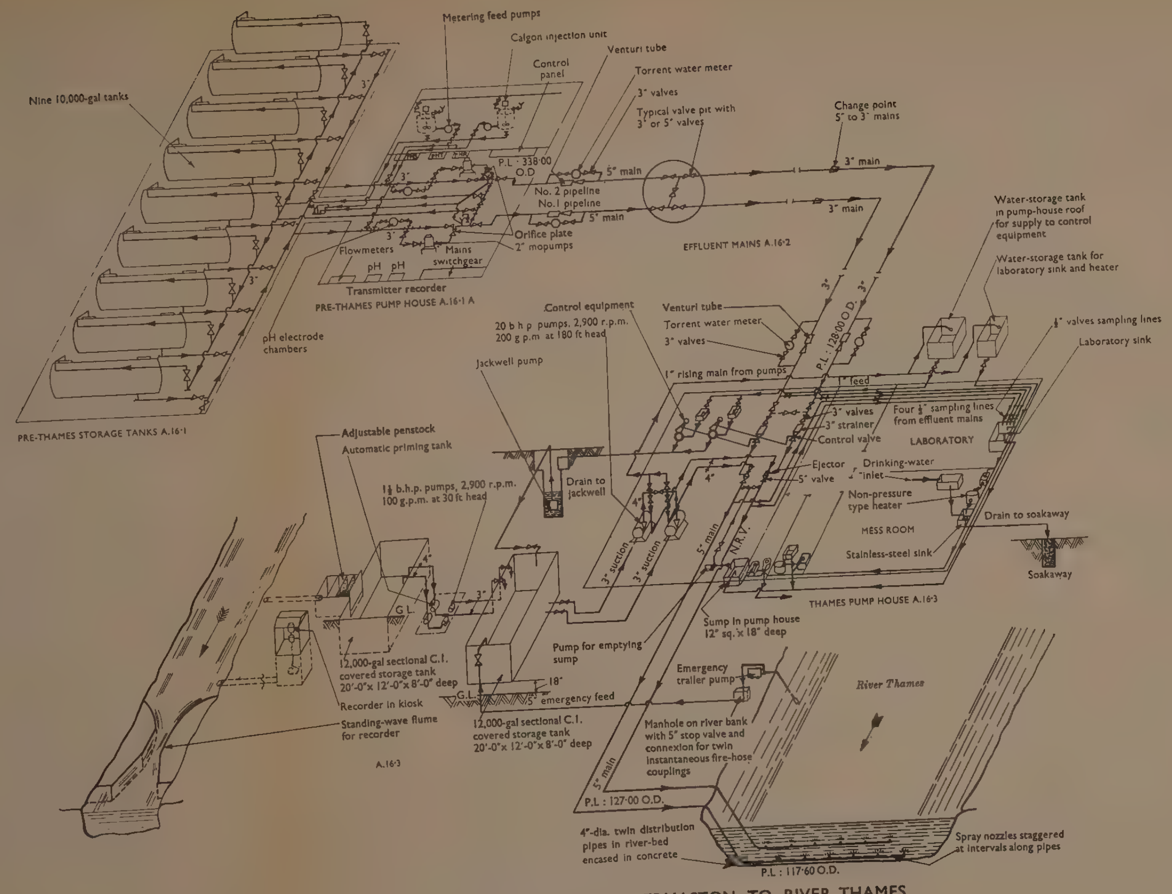

FIG. 15.—FLOW DIAGRAM FOR PIPELINE FROM ALDERMASTON TO RIVER THAMES

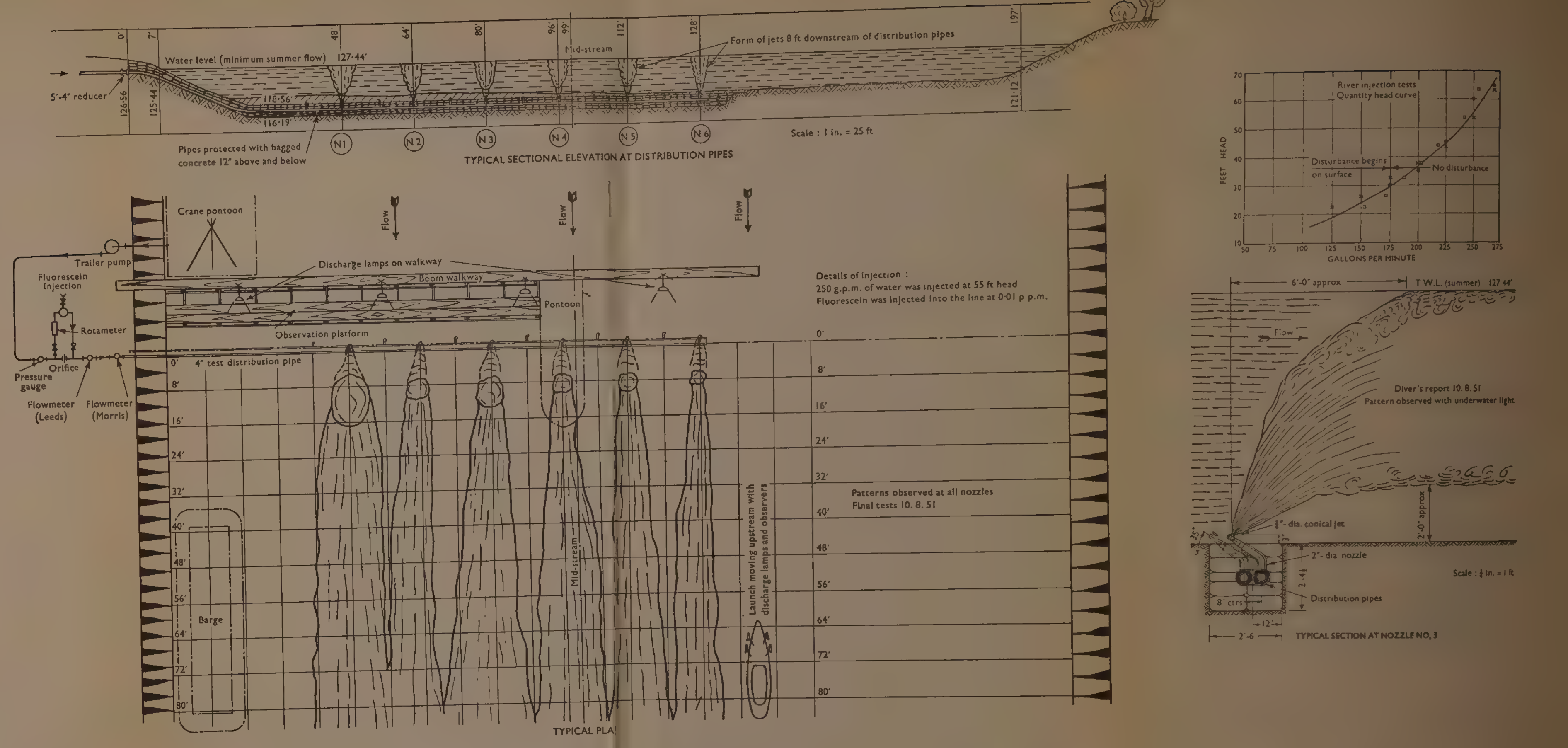

FIG. 16.—PLAN AND ELEVATION OF RIVER OUTFALL

WHITE and J. G. MILTON

William Clowes & Sons, Limited: London.

y 13 February, 1952, and both were in this condition fully tested and ready for ork on 10 March, 1952. In this period approximately 120,000 ft of main was laid ıvolving 6,000 welded joints, an 80 ft 0 in. river crossing, two major road crossings, nd a main-line railway crossing. Over a large proportion of the route water was ound at 1 ft 0 in.–1 ft 6 in. below ground level and as a consequence four well-point ets were continually in use and up to fifty mobile pumping sets. A maximum labour orce of about 400 men was employed using fifteen welding sets, the overall cost of the orks from the pre-Thames storage tanks to the river itself being approximately 400,000.

General remarks

The pattern for these works took shape in early 1950, construction reached its eak about August 1951, and completion was effected on 31 March, 1952, the cheduled day. The cost, based on an output of 25,000 g.p.d., an interest rate of $\frac{3}{4}$% for 15 years, and an allied sinking fund, totalled approximately £1,250,000 nd showed a capital charge of about 3½*d* per gallon of liquor discharged. Chemical nd labour costs hardly affect the figure quoted.

Obviously there is scope for new thinking on this subject in order to bring the harge down. No doubt, as the unknowns in the field of atomic energy become ewer, a more calculated approach to design will become possible and economies vill arise therefrom.

Acknowledgements

The Authors are indebted to the Director of the Atomic Weapons Research Establishment and to the Ministry of Works for permission to publish this Paper. There are many members of their staffs who made major contributions to the works lescribed to whom they are also indebted and amongst whom it would be invidious o select individuals.

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The Paper, which was received on 27 October, 1955, is accompanied by six photographs and seventeen sheets of drawings, from which folding Plates 1 and 2 and some of the Figures in the text have been prepared.

Discussion

The Chairman (Mr W. A. M. Allan) said that the disposal of radioactive wastes, which he thought the Authors would agree was still to a certain extent in its infancy, was of particular concern to public-health engineers from two points of view. One was that of those interested in the production of drinking water; at present radioactive elements were discharged into rivers and streams eventually used for water supply. The second was that of those interested in the disposal of sewage, who were particularly concerned with that problem, because disposal of radioactive wastes from hospitals, commercial laboratories, and industries would increase. As a result trade wastes which might possibly contain radioactive materials would be discharged to the sewers; although at the moment the quantity was small and the effects negligible, that might not always be so, and serious consideration might have to be given to that type of trade waste in the future.

He had been struck by what the Authors had said about collaboration. That was very important in all public-health engineering work and he had been very glad to read the Authors' statement that "many of the problems could best be developed by firms working in allied fields. . . . These firms were given basic principles to develop or, alternatively, were associated with current research". As a result of what might be called the working party so formed, which had included the manufacturers as well, the plant at Aldermaston was probably the best of its kind for the work which it had to do. Engineers who had to carry out such works often failed to take into consideration the vast experience which the manufacturers possessed in their own field, and which could be very useful.

He had been unable to find in the Paper any record of the daily quantity of sludge produced. That was of great interest to those concerned with sewage works. A great deal of research had still to be done in the sewage-treatment world to find a cheap and easy method of disposing of sludge. Engineers working in atomic establishments might find the answer to that problem. What had happened to the experiments carried out at Harwell on the freezing process for reducing the volume of sludge to be disposed of? Had they not been found practicable, or had the method been found too expensive?

Reference had been made to the necessity of diluting the effluent before it entered the river, and it appeared that mains water was used for the purpose. The Chairman was not familiar with the exact route of the discharge mains, but understood that they ran through or near a sewage works. Had consideration been given to the use of sewage effluent for the purpose?

Mr C. D. C. Braine (Partner, Messrs G. B. Kershaw and Kaufman, Consulting Engineers) said that in view of the considerable difficulties in the disposal of active sludge, the design of the sedimentation tanks was surprising. Each tank might have been expected to have a comparatively large central hopper in which the sludge scraped to it could consolidate before being drawn off; instead the Figures seemed to show a small hopper from which it would be difficult to draw sludge without "funnelling" occurring. The sludge, he thought, would not be liable to biological action; consequently it might be left in the hopper for 2 or 3 days before it was drawn off. On one job with which he had been associated some experiments had been made with ordinary sewage sludge to see what would happen. A rectangular tank with deep hoppers was used and when sludge was drawn off, for the first few minutes it had contained 89–90% of moisture, but after about 5 min, because the sludge did not slide down the hopper fast enough, water began to reach the outlet. Within 15 min, long before the hopper had been emptied, the sludge obtained had a moisture content of about 97%. If the sludge valve was then closed and the hopper was left for a quarter of an hour, the sludge began to level out again, and on re-opening the valve, sludge with a moisture content of about 92% was obtainable.

Because the sludge at Aldermaston was so light, he would have expected the sludge scrapers to be of the continuous-blade type with a peripheral movement far slower than was common in much larger tanks used for sedimentation of sewage. Even in those

ger tanks, peripheral speeds of 4 ft/min were about the limit if eddying round the raper blades was not to occur. The speed at Aldermaston seemed extraordinarily high. the basis of 4 ft/min the corresponding speed in the small tanks used at Aldermaston uld have been about 0·6 ft/min. Considering the lightness of the sludge, the speed at ich the scrapers were operated was about ten times what he would have expected.

The safety precautions taken everywhere on the Aldermaston plant seemed extreme. would be interesting to have an insurance expert's view on what premium should be id on the plant as designed, and the premium if certain precautions had not been taken. ere must be an economic limit beyond which it was unprofitable to go. He wondered ether he was right in supposing that at the present time all concerned with atomic stes were being extremely cautious and were therefore taking every precaution, but at probably in 10 years' time, when more knowledge was available, perhaps half the sent precautions would be dropped. From the point of view of a manufacturer using lioactive substances in a typical industrial plant, would it be practicable or possible in h a plant to take safety precautions on the scale adopted at Aldermaston?

During part of the 1939–45 war he had commanded a bomb-disposal company. For the t few weeks everybody had been extremely careful when handling bombs, etc., and all sonable precautions had been taken, but when the men had become accustomed to the rk, the problem arose of ensuring that they took proper precautions. If, when dealing h high explosives, the hazard of sudden death was not a sufficient deterrent to prevent elessness, what would happen when dealing with radioactive materials causing only a gering death, the signs perhaps not showing themselves for 10 years? In such cir-nstances, would it be possible to control the workers properly, or would it be necessary employ scientifically trained people who would appreciate the serious risks of care-sness?

At Aldermaston, the active sludge wastes were disposed of in steel canisters which, ased in concrete, were dumped into the sea. In that connexion, he had recently been ormed by an American delegate to the Nuclear Conference at Geneva that the problem lisposal of active wastes in America was regarded very seriously. Mr Braine had asked, re or less jokingly, what was wrong with the Atlantic for that purpose, and the reply l been that the Atlantic Ocean was not nearly big enough for what they expected to re to deal with shortly. The American had added: "You live at the receiving end of Gulf Stream, and moreover if we use the Atlantic we may contaminate the fishing banks und Newfoundland, so there is a limit to what we can dispose in the Atlantic". parently it was not going to be possible to use the sea or the land, so that all that was was the heavens! It had been suggested that the material would have to be shot up the sky and become a satellite going round the earth. What did the Authors expect lo when they had big quantities of active wastes coming from large plants in, say, 10 rs' time?

Mr W. Fillingham Brown (Consulting Engineer) remarked that he had not sufficient wledge to criticize the design of the plant in detail, but had been wondering if the hniques described in the Paper might be applied to ordinary sewage-treatment pro-ses, and particularly to those of chemical precipitation and mechanical flocculation. mical precipitation recently appeared to have suffered an eclipse in sewage treatment, whilst mechanical flocculation had been tried, it did not seem popular. The success he plant at Aldermaston did raise the possibility—although of course that plant was a very much smaller scale than most sewage-treatment plants—of applying those hods to sewage-treatment processes to improve sewage-works effluents, in view of the sent-day trend towards a higher standard of effluent. He thought there was a possi-y of a higher degree of removal of solid matter, perhaps by mechanical flocculation, or without chemical precipitation.

le had been interested also in the way in which the normally used and well-tried hods adopted at Aldermaston had been applied in what might be called a mechanical-ineering or chemical-engineering rather than in a purely civil-engineering manner.

The reasons were given in the Paper; one had been lack of time and another the necessity of detecting and containing possible leakages, but he wondered how the cost of maintenance would compare, over a period of years, with more orthodox civil-engineering applications.

The costs given at the end of the Paper were particularly useful, and the Authors were to be congratulated on including them, particularly because the plant was of a new type.

In the flocculation tanks provision was made for flow to be either upward or downward. How was it determined which way the flow should be directed, and did either give a better result than the other?

Would the high efficiency obtained—by ordinary standards it was of course extremely high, with a reduction of 99·9%—be maintained, particularly if the effluents varied at all from those at present, or was the type of plant at Aldermaston such that the Authors foresaw little or no variation in the type of effluent to be dealt with?

Mr R. W. Querée (Chief Engineering Assistant, Finsbury Borough Council) referred to some problems in the disposal of radioactive effluents, though not of the type described by the Authors or arising from the Paper.

There were many problems in municipal engineering right from the origin of the waste in a factory, laboratory, or other building, which might be situated in the centre of a town, and the municipal engineer had to deal with waste from its origin to its disposal. The Paper provided some thoughts on disposal, but the problems dealt with by the Authors would not arise in towns for some time to come; therefore perhaps they might be able to deviate from the subject a little and give some advice on the desirable limits of radioactivity so far as discharge to public sewers was concerned. That was causing a good deal of interest at the moment. He had in mind the discharge of active effluents through the normal drainage system, which might involve harm to occupants of buildings and to people in the streets from activity under the ground. It was also necessary to consider the men who worked in sewers, where the effluent was mixed with the sewage.

What concentrations did the Authors consider were permissible in a sewer, and could they also give some guidance on concentrations at treatment plants? He had seen published recommendations that the maximum should be 5 μc per flush and 50 μc per week per institution. He would also appreciate guidance on the storage of radioactive wastes in densely built-up areas. He had in mind factories which produced such wastes and wished to discharge them into the sewers. Some comments on the desirability of authorities being able to insist on storage or making provision for it would be very useful.

He would also welcome comments on the possible effect of those effluents on existing types of sewer. At Aldermaston, where the problems were special, stainless-steel pipes were used. Normally in sewage disposal salt-glazed stoneware or concrete were used for pipes. There had already been trouble in concrete pipes when acids were discharged. Problems were also met with in brick sewers. It seemed to him improbable that local authorities could ensure the removal of radioactive materials from effluents even from large factories.

Mr R. H. Burns (Chief Industrial Chemist, A.E.R.E., Harwell) said that on p. 626 it was not made clear that the tolerances given applied only to the Thames. In fact they were special figures, on an average one-hundredth of the internationally accepted drinking-water tolerances for occupational workers. The Thames figures had been recommended by the British Medical Research Council to afford complete protection to the public of London. The figures given were for the activity permissible in the Thames after any radioactivity had been added; they did not apply to the effluent itself.

The next point was that for the sake of accuracy the references in the Paper to the "Ministry of Health" should be deleted and "Ministry of Housing and Local Government" substituted.

On p. 627 it was stated that some of the drainage systems at Harwell were not fitted with control systems. To prevent any misunderstanding it should be emphasized that that was not now the case and all were now under strict control.

On p. 636 and subsequently the Authors had given figures for activity removal ranging om 93% up to 99·9%. It should be realized that that referred to alpha activity only. Where mixed beta activity was also present, as at Harwell, the efficiency of removal could ot be so high.

Referring to the use of demineralized water within the Establishment, Mr Burns sug-ested that one of the main reasons for that requirement was the much smaller bulk of udge produced during the chemical treatment of radioactive effluents. Since the emical sludges were disposed of at sea the cost of demineralization was soon recovered. was not clear to him how it was possible to measure alpha activity without evaporation s was suggested on p. 638 but possibly Mr White could explain the technique.

A statement made on p. 649 might give the impression that the discharges from Harwell nd Aldermaston were related to the extreme dry-weather flow of the Thames. That was ot so, as agreement had been obtained to use the harmonic mean flow of the river at the arious discharge points for dilution of the effluents.

On p. 651 mention was made of cathodic protection but there was no reference to the ossible use of magnesium electrodes. Had that been considered and if so, why had it een rejected?

Mr Burns hoped the Authors would not mind if he answered the Chairman's question n the freezing plant at Harwell. That plant had now been installed and the acceptance sts had been carried out. They had given very favourable results and although the lant was in its early days it appeared that a 50% reduction in sludge volume could be xpected. If that was realized, even ignoring the great increase in filtration rate and ther advantages, the saving involved would pay for the plant in about 12 months.

Mr S. H. Lewis (Senior Engineer, Messrs G. B. Kershaw and Kaufman, Consulting ngineers) said he would like to ask the Authors about their views on time factors. It ad been stated that the plant in question had been built to permit important atomic xperiments to proceed, only 5 years ago. (As an aside, he would suggest that the omic bomb was the devil that drove.) A period of 10 years had been suggested as that uring which industrial uses of radioactive materials might become far more widespread, nd as a result there would be industrial effluents which would contain intolerable amounts f active material. Should the excellently designed plant at Aldermaston be regarded as pilot plant for one which might have to be produced on ten times the scale within an verage lifetime? If so, were materials such as stainless steel and the alloys referred to bsolutely essential? It appeared that cost of effluent disposal would be a limiting factor that were so.

It had been emphasized that water-treatment plant technique was the basis of the uccessful production of the effluent discharged to the Thames. That seemed to him sential, because having regard not only to Aldermaston but to other similar establish-ents, he imagined that the build-up of radioactive material in streams which were ready, or became, sources of potable water must be a serious problem. That brought m back to his original question about the time factor. For how long would it be possible tolerate such a build-up as would arise from peaceful, industrial uses, or other uses, hich produced similar effluents?

Dr K. J. Ives (Lecturer, Department of Civil Engineering, University College, ondon) said that most of the speakers had shown their interest to be in relation to their perience of sewage works. His own experience lay in water purification and in the eration of three different types of upflow units, similar in principle to those employed Aldermaston. He wished to speak on sludge treatment. On p. 636 the Authors had id that "a high price can be afforded for treatment chemicals which give an appreciable duction in sludge volume". Increasing use was being made in the water industry of a emical-activated silica, which reduced the quantity of sludge to be treated. It added the cost, in that it was a reagent, but the reduction in volume it brought about might ake that extra cost worthwhile. In his own experience, which had been with the use of

aluminium sulphate, the use of the activated silica as a coagulant aid had reduced sludge volumes quite considerably with three types of clarifier.

He could give some typical figures. A dose of 60 p.p.m. of alum in an Accelator gave an average volume of sludge of 0·9% of the water treated. When 10 p.p.m. of activated silica was added, that was reduced from 0·9% to 0·4%, or just under half. Operating with a cylindrical clarifier, with a dose of 100 p.p.m. of alum, the volume of sludge had been 6·9%, and by adding activated silica that had been reduced to 3·3%, or again just under half. On another occasion, when the water temperature had been different—which affected the volume of sludge—4·4% with alum alone had been reduced to 0·6% by the use of activated silica, which was quite a substantial reduction.

Lastly, using a hopper-bottom clarifier, similar to that described in the Paper, with an alum dose of 20 p.p.m. the sludge volume was reduced from 3·8% to 1·2% by adding 2 p.p.m. of activated silica. Had the Authors considered the use of activated silica when discussing the use of aluminium sulphate coagulation at the outset of their experiments? He did not know whether activated silica had any application to the tannic-acid-lime process, and would be glad of enlightenment on that point. An additional advantage of the silica was that it gave mechanical strength to the floc, which was less likely to be broken by turbulence or shear in the water. That might be of advantage in the Authors' plant.

In the operation of a conventional water-treatment plant he had found it an advantage to pass the sludge from the clarifiers first to sludge-settlement lagoons, where the sludge fraction settled under completely quiescent conditions, with no flow, and the supernatant water could be pumped back for retreatment and the thickened sludge then taken away for further treatment.

Mr J. P. Asquith (Manager, Engineering Division (Projects), W. J. Fraser & Co. Ltd) said that in the Paper there were two references to the Harwell experience: first, where contamination had occurred in streams where it had not been expected, and secondly, concerning effluent limitation where the various devices of recirculation and limitation on water usage were mentioned. It seemed to him that basically the less effluent produced in the first place, the easier the problem was made in magnitude throughout. He would like the Authors to elaborate a little therefore on just how far it was possible to go in the reduction of the total size of the problem by a bigger internal re-use of water so that the problem never got outside.

Mr F. E. Ireland (Inspector of Alkali, etc., Ministry of Housing and Local Government) wrote asking whether the type of tannic-acid-lime precipitate was important for the efficient removal of the radioactive constituents. In other words, would a denser more crystalline precipitate be just as effective as the light flocculent precipitate now being obtained?

Did the precipitation conditions follow von Weimarn's theories of precipitation and were they taken into account when designing the treatment plant? In general, particle size was proportional to the degree of dilution of the reactants and inversely proportional to the viscosity. Regarding dilution, could a denser precipitate be formed by back-mixing some of the treated effluent from, say, the flocculent tank with the treatment chemicals before addition to the flume mixer, or better still, by designing a continuous mixing vessel whereby the reactants were added at diagonally opposite points and received adequate dilution by the product before they met? Viscosity could be lowered and settling aided by heating the reactants, and he wondered whether waste steam was available for keeping the effluent warm in the pretreatment storage tanks. Using those techniques it was possible to produce crystalloid-type precipitates of greater bulk density and holding less entrained water than many gel-like precipitates formed by more conventional methods of precipitation.

Mr White, in reply, said that reference had been made in the Paper to the consideration given in the design to the probable use of citric acid as a decontaminant. So far as

heir knowledge had gone at that time, citric acid was the most likely decontaminant to be ised. However, the Paper would become out of date if it were not stated that much esearch had been since done on decontaminants, and far more modern methods were now ised. Nevertheless, the principle of having separate collection lines for liquors which ontained decontaminants was sound.

A point perhaps not sufficiently emphasized in the Paper was the degree of automatic ontrol possible in the chemical-treatment plant. Provided that all the services were in peration—chemical, electrical, and demineralized water—it was possible by a single throw f a switch to start a complete section working. Automatic control was optional, and the lant could be operated manually at all points, either if the automatic system failed or for esting individual units.

Control was exercised over the movement of effluent to ensure that a number of checks vere made between discharge from buildings and discharge from site, and so that any articularly high activity discharge from a building could be specially segregated or treated istead of being allowed to raise the general activity level of the plant. The delay tanks utside individual buildings were sampled and analysed for radioactivity before a "Move" ertificate was issued, and no pumping took place from such tanks until the certificate was ssued. Much complicated paper work was involved, but they hoped in the near future o speed it up by the installation of a facsimile transmitter from the laboratories to the novement-control office. At the bulk-storage area a similar procedure of radioactive nalysis before authorizing movement was adopted and the level of activity determined.

At the pre-Thames-storage stage a sample was taken and divided into five parts. One vent to their own laboratory for radioactive check, a second to their own laboratory for he ordinary trade-wastes check, a third to the Establishment's Health Physics Branch, he scientists of which were finally responsible for ensuring that the activity level was atisfactory. The fourth went to the Ministry of Housing and Local Government for a adioactivity check and the fifth was kept in the Establishment as a reference sample. 'inally, continuous samples were taken at the point of discharge to the Thames and hecked for radioactive content and for the way in which they met normal trade-waste equirements.

The Aldermaston plant had now been in operation for about 4 years, and it might be of iterest to consider the extent to which the performance of the plant had come up to esign expectations. First—and that partially answered one of the questions raised in the iscussion—the plant had handled successfully all radioactive effluents from the Estab-shment without once exceeding the permitted tolerance level—and there had been a ariety of types of effluent certainly not expected originally, including even small amounts f beta and gamma effluents, which, as Mr Burns had pointed out, could not be treated 'ith the same high efficiency in such a plant.

Total activity discharged to the Thames had been no more than one-third of that cceptable to the authorities, even though the basis of the tolerances was extremely con-ervative. The plant's efficiency had been maintained at 99·9% whenever the activity ad been high enough to need that, although the normal level of activity discharged was ery low and a working efficiency of about 99·7% was usually obtained.

The average sludge volume to be disposed of at sea was about 0·2% of the treated ffluent, appreciably better than the figure obtained in pilot-plant studies. That probably nswered the Chairman's question about the quantity of sludge disposed of. Mr White ad done a rough calculation based on the amount treated, and using that figure, it came o 15 gal/week. That might also answer the question on the smallness of the hopper in ne bottom of the thickeners, because the amounts of sludge with which they were con-erned were extremely small, and the small pits at the bottom of the clarifiers held a eek's sludge or even more. In the thickeners, unless care was exercised, the sludge ecame so thick that it could not flow reasonably into the can unit. It seemed, therefore, nat no modification was required so far as the final collecting was concerned.

The total rate of effluent discharged had gradually increased until it now averaged),000 g.p.d., which was getting near the calculated discharge rate given in the Paper of

25,000 g.p.d. In general, therefore, it would be agreed that the actual performance had matched the design expectations.

Mr Burns had answered the Chairman's question about the freezing process adopted at Harwell for reduction of the volume of sludge. At Aldermaston an alternative method of reduction of the sludge by evaporation had been developed, in which the cans in which the sludge was discharged were taken and use was made of the very efficient ventilation system to evaporate some of the water, to increase the solid content, which was only about 9%. One drum could therefore take a successive number of discharges of sludge, so that the total volume of material to be disposed of was considerably reduced.

On the question of to what extent the precautions at present taken in the plant might be dropped now that more experience of radioactivity had been gained, Mr White hesitated to give an answer except in very general terms, because it would be appreciated that it was a subject on which the Atomic Energy Authority was constantly engaged. They could not spend public money to any greater extent than was essential. It was mainly a question of engineering to the tolerances clearly laid down by the medical experts. If they were told that certain concentrations were allowable in the air and certain others in the water, it was only straight engineering to ensure that those tolerances were observed. They had had experience in the Atomic Energy Authority of where, if precautions were relaxed only slightly, they began to get far too close to those tolerance levels. They found very often that they were working to a nice balance in just ensuring that they met the figures laid down by the health authorities.

It had been asked whether the efficiency of the plant was being maintained. He had already mentioned that an efficiency of 99·9% was being maintained whenever that was required.

Mr Burns was right in suggesting that the emphasis on the use of demineralized water was a matter of reducing as far as possible the amount of sludge obtained from the water-treatment plant. All the dissolved solids in the water were likely to come out in the sludge. The point they had intended to make was that the accuracy of measurement by the evaporation method was lessened if there was a large quantity of solids which came out in the evaporated sample, and that accurate control was therefore more possible with demineralized water being generally used.

They had toyed with the idea of using activated silica, but having regard to the very small quantities of chemicals used they had concluded there was no likely advantage from its use, although they were well aware of its advantages. They were, however, very interested in the figures advanced by Dr Ives; that was a point which might deserve re-examination.

In regard to Mr Ireland's remarks, although little was known about the effect of the physical type of precipitate on radioactivity removal, Mr White would not anticipate that a more crystalline precipitate would be a disadvantage.

The use of dilution as a means of achieving that, however, would not be likely to be successful as radioactive removal was certainly less efficient at lower concentrations. Heating the liquid to reduce the viscosity had been considered, although there was no waste stream available, and had been dismissed partly on cost grounds but also because it had been found that minor temperature variations (if only a few degrees) in the flocculating vessels were sufficient to cause convection currents which seriously upset the settlement of the floc.

Mr Milton, in reply, said that it had been evident that there were water engineers present at the discussion who, he was sure, were very appreciative of the use made of the orthodox pattern of water treatment. They knew the difficulties in treating the complexity of highly alkaline coloured water, inasmuch as very great care had to be taken with pH control when forming the floc, and the measures necessary to preserve the floc from its formation to its subsequent settlement or deposition on filters. The handling of the floc had been a major problem in the design of the Aldermaston effluent-treatment plant. The Paper indicated that the rate of treatment had been considerably slowed down from the orthodox pattern. In the orthodox method the time for settlement after

hemical treatment varied according to the content of the raw water from 1 to 2 hours, ut at Aldermaston settlement took 10 hours. Again, the orthodox speed of filtration as about 80 gal/sq. ft/hour, whereas at Aldermaston it was 26 gal/sq. ft/hour. The normal elocity through pipes in water-treatment plants was 2–3 ft/sec, whereas at Aldermaston hat velocity was about 0·25 ft/sec.

Ways and means would obviously have to be considered on future plants to reduce the ost of treatment per thousand gal. The experience gained from Aldermaston would, it as hoped, give an indication of the avenues along which that could be brought about. Ie felt that the present Paper, and the two earlier Papers (references 2 and 3 on p. 655) ould be of considerable assistance to many engineers who were bound to have to deal with adioactive effluent problems as the scope of atomic engineering widened in the industrial eld. In answer to Mr Allan's question on the use of sewage for dilution, it could only be aid at the present time that provided that the River Board concerned would accept the ewage effluent into the river there appeared to be no objection to the effluent being used or that purpose, the qualification being that it would have to be entirely free from uspended matter. However, that was a subject for further research work.

Mr Wilson, in reply, said that collaboration with industry had been undertaken for he great specialist knowledge which the Ministry did not possess. It was to be a wise echnique.

The dilution effected prior to discharge was purely to increase the mass of water entering he river in order to effect good distribution in it. The effluent rate was about 50 g.p.m., nd to distribute that quantity in the river would have meant small nozzles or some imilar technique which might have been affected by river silt.

They had given serious thought to the limitation of costs of effluent-treatment plant. Iowever, Harwell, Aldermaston, and Windscale were the first plants of their kind, and it ould be very foolish to hamper a new science by having an accident through not being areful. They had therefore erred on the side of safety, and he thought that they had een right. Nevertheless, they did continually in that and other fields, when opportunity ffered, derate materials and there had been a movement from stainless steels, of which here had appeared to be a need in the early days, to plated and painted mild steel. They id not derate, however, at the expense of running any risk.

The disposal of materials at sea was a subject on which he could not speak with authory, and he must leave that to others.

The Authors had quoted advisedly the operational costs given in the Paper. They ould not be said to be completely accurate; perhaps they should be modified a little ecause obviously in the figures quoted there were acceleration costs, because the work ad had to be done quickly. There was also rather a long pipeline, which not every plant ould require. The quoted design figure per gallon of effluent was related to less than 24 ours per day, so that the price per gallon could be reduced. It would still be a high price er gallon, however, and that was the point which they wanted to bring out.

Mr Querée had raised some very interesting points on the subject of tolerances in public ewers and the like. Mr Wilson did not feel himself competent to comment on those sues. Much more would of course be published on the subject as time passed. Mr uerée's points about the effect of radioactivity on pipes and the use of various materials as a subject on which a great deal could be written.

The Authors were glad that Mr Burns had been able to take part in the discussion and ut the question of tolerances in its right context. They would all be very wise, when they ead of a tolerance in such a Paper as the present one, not to think of it merely as a figure ut to have regard to its background. Mr Wilson was also glad that Mr Burns had made clear that there had not been any failure at Harwell on the non-monitored pipeline. he deficiency in the system had been dealt with before any trouble arose, as he believed as mentioned in one of the previous Papers.

Mr Asquith raised the question of the conservation of water to reduce the quantity of fluent so far as possible. The techniques possible for that purpose depended on the type

of process in use. Whenever they were dealing with a prospective radioactive effluent they went to considerable lengths to reduce the quantities to the absolute minimum, and when the plant was working the operators did all that they could to achieve the same end.

Finally, the plant had been designed against the particular needs of the Aldermaston establishment. It should not be assumed that such plant was a necessity at every atomic energy establishment; indeed for the atomic energy power stations now being built very little was required, inasmuch as the radioactive arisings would normally be small.

The important aspect of the Paper was the fact that relatively normal water-treatment plant could contain and precipitate active effluents of the type under review.

HYDRAULICS DIVISION

JOINT MEETING WITH THE INSTITUTION OF MECHANICAL ENGINEERS

1 May, 1956

Mr G. A. Wauchope, Past-Chairman of the Hydraulics Group of the Institution of Mechanical Engineers, in the Chair

The following Paper was presented for discussion and, on the motion of the :hairman, the thanks of the Institutions were accorded to the Author.

Hydraulics Paper No. 10

PUMPING PROBLEMS—PRESENT AND FUTURE

by

Herbert Addison, O.B.E., M.Sc., M.I.C.E., M.I.Mech.E.

SYNOPSIS

In moving large volumes of water, it seems likely that purely gravitational schemes lone will no longer suffice; in many instances pumps will increasingly be needed. One im of the Paper is to examine this trend, taking as examples the application of pumps in ressure-boosting installations, in hydro-electric works, and in land reclamation.

In a more general way the Paper tries to show the interdependence of large pumps and ıe civil engineering works with which they may be associated, i.e., pipes and conduits ıould take into account the particular characteristics of the pumping machinery. Imroved types of pumping plant, however, may directly facilitate civil engineering conruction, e.g., in the "well-point" system of arresting the flow of underground water ıto excavations.

One future problem touched upon is that of expanding world food supplies to keep pace ith the expected rise in population. Irrigation pumping plants, probably working under lverse conditions, will play their part here. In this connexion the Paper discusses the ılp that might be received from electrical-distribution systems fed from atomic-power ations. At present, such sources of energy do not seem economically attractive in ›mparison with existing sources such as oil or water power.

As an illustration of a large-scale irrigation project depending upon numerous small ımps fed by hydro-electric stations and using underground water, one of the Indian tube-well" schemes is mentioned.

INTRODUCTION

HERE are at least three reasons why, at present, the civil engineer could advantageısly survey anew the whole field of water-pumping problems. One is that since ırface-water resources are being exploited more and more intensively, the engineer ıs to look with increasing diligence for underground supplies, and underground ater must be pumped to the surface. A second reason is linked with the need to ıgment agricultural production to keep pace with an expanding world population. ı certain regions it may be found that the only new and hitherto unproductive

areas that can be put under irrigation are high-lying, to which water must be lifted mechanically. Probably, too, the pumps will have to work in adverse conditions, for if local circumstances had been attractive the area might already have been developed. On the other hand, such disadvantages might be neutralized if a new source of energy could be made available on very favourable terms. It is this possibility that suggests the third reason for examining pumping potentialities; it prompts the question, what role may atomic-energy power plants be expected to play?

More generally, the problem as a whole could be expressed as a trend, a trend away from the province of the civil engineer into that of the mechanical engineer. The movement of bulk supplies of water, for example, will depend increasingly upon pumps rather than upon the impulsion of gravity. Some typical examples of this trend could be grouped under four different headings, such as:—

(1) Pressure-boosting installations.
(2) Pumps in association with hydraulic turbines.
(3) Pumps in relation to other engineering projects.
(4) Pumps for augmenting agricultural production.

Although all these selected examples may not show the mechanical engineer actually coming to the rescue of the civil engineer, they may illustrate the advantage of closer co-operation between the two. Few of the problems depend for their solution upon specifically new designs of pump. Existing forms of rotodynamic pump—centrifugal, propeller, or screw—can nearly always be adapted. What is really essential is the ingenuity and resourcefulness required to choose the right size of pump, the right number of pumps, and the right place to put them. In contrast to gravitational schemes, in which the terms "dead head" and "static head" are almost ominously descriptive, pumping projects might be regarded in terms of liveliness, elasticity, and adaptability.

Pressure-boosting installations

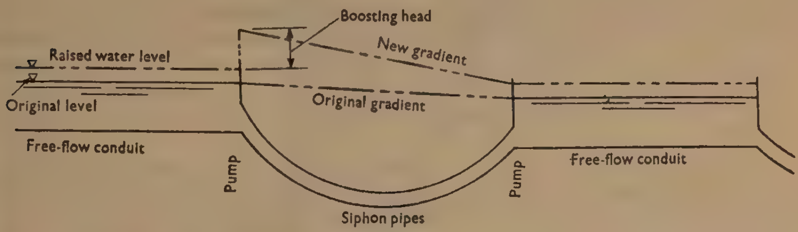
Basically, the aim of a pressure-boosting installation is to make more water flow along an existing pipeline.[1] This can be done only by increasing the slope of the hydraulic gradient. If the gravitational head on the system has already been fully exploited, a booster pump interpolated in the pipe can generate an additional head, thereby steepening the hydraulic gradient and giving the desired additional water supply. In one aqueduct alone it may be found that at different periods changed circumstances have imposed different solutions. This has happened in the operation of the Thirlmere aqueduct of the Manchester Corporation waterworks. Originally designed to conduct water by gravity alone from the Thirlmere reservoir to Manchester, a distance of 96 miles, the aqueduct took the form of free-flow "cut-and-cover" conduits, alternating with siphon pipes under pressure. Between 1894 and 1928, demands for more water could be met by progressively increasing the number of siphon pipes working in parallel, so that at the end of this period four such pipes extended as far south as Lostock, $12\frac{1}{2}$ miles north-west of Manchester, and three pipes only for the remaining distance.

At this stage a new system came into effect. Instead of laying the remaining length of the fourth pipe onwards to Manchester, a boosting plant was built at Lostock. As in all such installations, the pumps imposed the necessary artificial

[1] The references are given on p. 699.

ead on the water in the existing three pipes, and thereby enabled them to carry as uch water as the four pipes in the remainder of the aqueduct. Moreover, the gross uantity flowing could be controlled more easily, first by varying the speed of the lectrically driven centrifugal pumps, and later by adding further pumping units. y 1932 there were three sets of pumps at Lostock, each consisting of four pumps, nd they could impart a maximum boosting head of 120 ft.

When, after the second world war, still further demands were made on the system, e Lostock installation could give no more help. No matter by how much its apacity might have been augmented it could not have induced more water to reach anchester; it could not have appreciably drawn down the hydraulic gradient along he 83½ miles of aqueduct between Thirlmere and Lostock. It is true that in the ee-flow sections an increase in the water depth would have given an augmented ischarge, but in the pressure pipes themselves the equivalent increase in velocity ould be achieved only by steepening the hydraulic gradient *in each individual pipe.* n principle each siphon pipe should have its own booster pump (Fig. 1).



IG. 1.—ORIGINAL AND MODIFIED HYDRAULIC GRADIENTS IN THIRLMERE AQUEDUCT

In earlier years no economically practicable solution might have been found to uch a problem, but by 1946 progress in the design of rotodynamic pumps made it ossible to resolve the difficulty quite elegantly.[2] As Fig. 2 indicates, inclined-type ropeller pumps were used, installed in the existing northern or inlet well of each iphon; they replaced the inlet bellmouths through which water originally flowed by ravity. A two-speed gear box interposed between electric motor and pump permitted two degrees of "boost" to be chosen to suit the desired increase in discharge. This imposed pressure head in different parts of the system ranged from 2 to 18 ft, nd in this way the discharge in the Thirlmere aqueduct as a whole was raised from 6 m.g.d. to 53 m.g.d. The whole system embraced fifteen boosting stations, in which a total of seventy-three pumps was installed.

One of the reasons that intensified the need for these emergency pumps in the Thirlmere aqueduct was unavoidable delay in completing another of the Manchester waterworks projects; this was the Haweswater reservoir and aqueduct. Although ts ultimate yield was assessed at 105 m.g.d., yet, at this time, in 1946, none of this water could reach Manchester, and because of post-war difficulties of various kinds, here was no prospect of the aqueduct being ready for several years. Nevertheless, at one point the aqueduct ran so close to the existing Thirlmere aqueduct hat a cross-connexion between them could be contrived. In this way water from he already completed Haweswater reservoir could be diverted into the Thirlmere queduct and could thereby profit from the newly installed booster pumps.

At the time of writing, 1955, the Haweswater aqueduct has been successfully

completed and is delivering water direct from the Haweswater reservoir to Manchester. But this lightening of the burden on the Thirlmere system does not mean that the utility of the booster pumps has been exhausted. On the contrary, they now have important new duties. The abundant supplies now coming from Haweswater make it possible from time to time to take the Thirlmere aqueduct out of service and to carry out extensive repair work to the concrete free-flow sections. During the intervening periods when the line is put into service again it must give its maximum possible discharge to maintain the stipulated overall annual flow. This can be done only by running the booster pumps at their maximum speed. These varied experiences in the Manchester waterworks system do illustrate, then, the elasticity and adaptability of operation of auxiliary pumping plant.

A comparable situation at Cape Town, South Africa, was met by a more orthodox boosting installation. Until 1942 the water supply to the city flowed by gravity

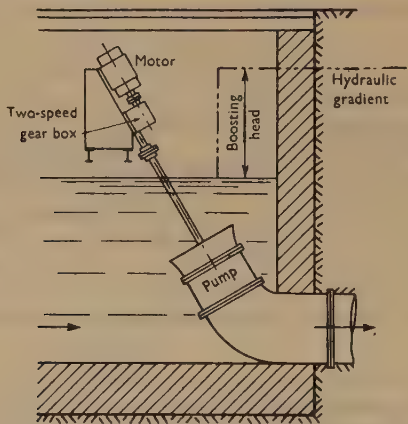


FIG. 2.—BOOSTER PUMPS IN THIRLMERE AQUEDUCT

from the reservoir at Steenbras along a 32-in.-dia. main $32\frac{1}{2}$ miles long. But even before the outbreak of war the gravitational discharge of 11 m.g.d. had been found inadequate, and now an additional emergency had arisen. Convoys between Great Britain and the East were at that time routed round the Cape, and when they called there the ships required large quantities of fresh water. Fortunately two booster pumps, ordered some years previously, were by this time ready for service, thus boosting the discharge through the pipeline to a maximum of 15·5 m.g.d. Because of the need for simplicity and speed in construction, automatic control of the pumping sets was not thought necessary; both the speed of the pumps and their disposal singly or in series were under hand control.

But like the Thirlmere aqueduct pumps, those at Cape Town were still found serviceable after the passing of the emergency that had called them into being. By 1949 an additional 33-in. gravitational main from Steenbras to Cape Town had been laid parallel to the original main, yet the booster pumps still continue to run for about 6 weeks during the summer period.

Pumps as auxiliaries to hydraulic turbines

In conventional high-head water-power installations all the water delivered to the ydraulic turbines flows by gravity alone and, as a rule, the flow from one particular atchment is collected in its own storage reservoir. Yet because this reservoir, and he dam that constitutes it, may form one of the most costly items of the whole roject, and since their location is controlled by rather intractable considerations of opography and geology, there is every inducement to make the fullest possible use f them. One way of augmenting the utility of the system is to divert into the eservoir water from other and adjoining catchments which could not otherwise be conomically exploited, the water flowing by gravity along race-lines or through unnels. This procedure is used to advantage in some plants of the North of Scotland Iydro-Electric Board.

Even if these auxiliary sources of water originate at a level below the top water evel of the main reservoir, they may still be utilized. The water may be pumped p into the reservoir. The net energy thus expended is recovered in the main nstallation, and although conversion and frictional losses are inevitable, the overall ffectiveness of the system may be greater than if the water had been exploited in a umber of small auxiliary power plants.

In various other ways, too, pumps and water turbines may work in harmony, as he following examples show.

ake St. Clair installation, Tasmania

Water for the Tarraleah station of the Hydro-Electric Commission of Tasmania was riginally stored in a natural lake, Lake St. Clair.[3] It was so deep that only a rela-

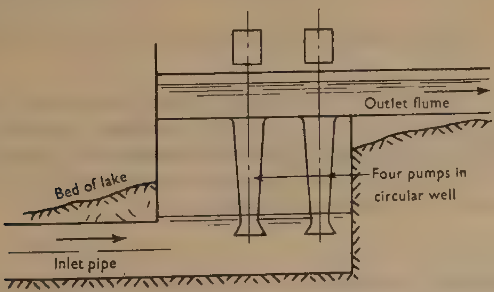


Fig. 3.—Lake St. Clair pumping installation, Tasmania

ively small proportion of the gross capacity of the basin was available by gravita-ional flow alone; most of the water lay below the level of the natural outlet of the ake. Improvements of two kinds were therefore effected; a weir across the outlet aised the maximum water-surface level, and a pumping station enabled much of the vater to be drawn off that would not flow out by gravity (Fig. 3). In this way a torage capacity of 117,000 acre-feet was added to the gravitational storage of 36,000 cre-feet.

Constructed on the bed of the lake, the pumping plant comprised four vertical-shaft ropeller pumps each having a normal discharge of 150 cusecs, and each driven by a 00-h.p. electric motor. The head against which they work, which ranges from 8 to

28 ft, is insignificant in relation to the gross head of about 1,000 ft which is available for the hydraulic turbine units.

Normally the pumps run only during the summer months, but during a recent drought in Tasmania they have been kept working continuously at full capacity.

Tarraleah pumped-pondage scheme, Tasmania

On its way from Lake St. Clair to the turbine forebay at the Tarraleah installation, all the water originally flowed along an open flume 13 miles in length, the capacity of which—900 cusecs—limited the turbine output to less than 100,000 h.p., yet the installed capacity of the station was 126,000 h.p. As the load on the system grew, there arose a need to utilize the full generating capacity during a short peak period each day. Whilst civil engineering improvements elsewhere made progress, a mechanical-engineering expedient offered a quick temporary solution. At the end of the flume, near the forebay, a small auxiliary reservoir was contrived; it was filled during the night hours by pumps drawing from the forebay, while during the peak generating periods water was released from the reservoir to augment the discharge flowing along the flume (Fig. 4). Each of the three horizontal-propeller pumps had a capacity of 30 cusecs against a maximum head of 20 ft.

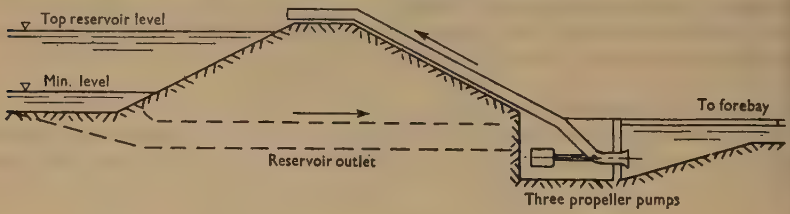


FIG. 4.—TARRALEAH PUMPED-PONDAGE SCHEME, TASMANIA

After several years of successful operation these pumps have now fulfilled their purpose and have been removed for service elsewhere. Another open flume, in parallel with the first but at a slightly higher elevation, now ensures that all water in this part of the system can be brought to the turbines by gravity alone. Nevertheless, the auxiliary balancing reservoir is still useful.

Pragnères pumping scheme, Pyrenees

This installation directly embodies, on a major scale, the principle of lifting various secondary sources of water into one main reservoir.[4] Here, on the northern slopes of the Pyrenees, the main artificial storage basin has a capacity of 67,000,000 cu. m; its top water level is at elevation 2,160 m and, under a head of 1,150 m, it feeds two Pelton wheels having a gross output of 200,000 h.p. There are two auxiliary pumping plants (Fig. 5). The one at La Gloire works against a head of 140 m. and its two pumps each have a capacity of 0·85 m^3/sec; at Pragnères, where the head is 400 m, two of the pumps each discharge 2 m^3/sec, and one pump discharges 1 m^3/sec.

These figures show that the combined volumetric pumping capacity is quite substantial in relation to the maximum demand on the turbines; it is more than one-third. As for the energy input to the pumps, the largest of the units have driving motors rated at 14,000 h.p. each.

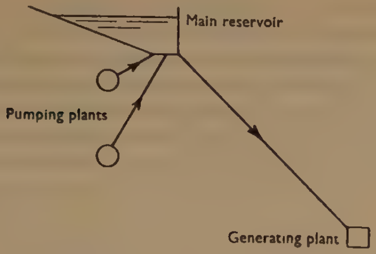


Fig. 5.—Pragnères auxiliary pumps, Pyrenees

'olorado–Big Thompson project, United States

. In this American scheme, pumps ensure that water from the western slopes of the ‹ocky Mountains can be used to generate hydro-electric energy on the eastern slopes Fig. 6). Although in principle a wholly gravitational system would have served, he diverted water flowing eastwards through a tunnel, yet there were advantages in

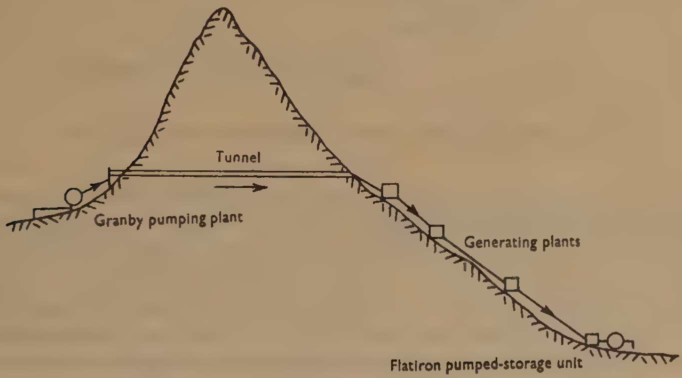


Fig. 6.—Colorado–Big Thompson project

ing auxiliary pumps.[5] The length of the tunnel would thereby be substantially duced, and useful storage capacity on the head waters of the Colorado River could contrived. From the lower of the two basins, the artificial Granby reservoir, the ımps lift water into the upper reservoir comprising the natural Shadow Mountain ıke and Grand Lake. The water then enters the tunnel, 13 miles long and of ft 9 in. dia. There are three pumps in the Granby plant, each of 6,000 h.p. ıd each capable of lifting 200 cusecs against a head which may range from 96 to 6 ft.

After emerging from the eastern end of the tunnel the water descends through a tal height of 2,900 ft, passing in turn through four hydro-electric plants having an gregate output of 200,000 h.p. Finally, some of the water is discharged into a twork of irrigation channels, and thus supplements irrigation over an area of 0,000 acres in north-east Colorado.

14

Pumped-storage installations

Although pumps and water storage have been associated in all the projects just described, the projects could not on that account alone be described as "pumped-storage installations." The term nowadays generally has a more specific significance. It is used to describe an assembly of pumps, hydraulic turbines, and reservoirs that together form part of a larger interconnected distribution system, their purpose being generally to adjust differences between overall electrical energy supply and energy demand. When the call for energy is less than the available supply, as it might be at night or at week-ends, surplus electrical energy from the other stations of the system can be used to drive the pumps of the storage station and thereby to accumulate hydraulic energy in the high-level reservoir. This stored energy could be released at times of peak demand. As the water flows downwards through the hydraulic turbines, electrical energy is generated which supplements the supply from the main stations.

A conventional high-head hydro-electric scheme with its own reservoir can be given auxiliary-storage facilities by adding pumping plant as in Fig. 7. Here the

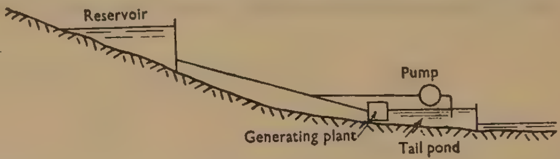


FIG. 7.—HYDRO-ELECTRIC INSTALLATION WITH STORAGE PUMP

pumps would serve as a kind of insurance premium against a season of exceptionally low rainfall or river flow, which might deplete the reservoir so seriously that the main hydraulic turbines could no longer deliver their specified output. If there was a tail-pond or downstream reservoir from which the pumps could draw, they could utilize off-peak energy from elsewhere in the system and replenish the main reservoir. It is conceivable that in this way the desired continuity of service of the main hydraulic turbines could be ensured at an overall cost less than that of heightening the dam and increasing the main reservoir capacity. Alternatively, if the hydro-electric station was itself adapted for peak-load operation, its energy output could be increased by installing pumps. At night they could lift back into the main reservoir some of the water used during the day.

In what might be termed a pure pumped-storage plant, the entire installation is wholly separate physically from the other stations in the system (Fig. 8). Virtually the whole of the water that descends through the turbines from the upper to the lower storage basin is returned by the storage pumps. The installation contributes no net energy to the system in its entirety; indeed it dissipates energy. Because of conversion and friction losses of various kinds, perhaps not more than two-thirds of the total energy received by the pumped-storage plant is usefully returned to the system.

To show how the installation may nevertheless be economically attractive, let it be assumed that a group of thermal stations is the original source of energy.[6] If these stations were dependent upon themselves alone they would need a gross available capacity at least equal to the gross anticipated peak-period demand. If then the demand continued to rise it could be met only by constructing new and necessarily

ostly thermal stations. If, however, a pumped storage plant was built instead, the ydraulic turbines would take care of the additional peak-load; moreover, this peak-ad energy would command a high price. On the other hand, the replenishment f the upper storage basin at night can be done by low-cost energy. In this way the verall load factor of the existing thermal stations would be raised, with a corres-onding gain in overall economy. Other advantages might accrue also. For dealing rith rapid changes of load the hydraulic turbines might be more convenient to perate than the generating units in the thermal station.

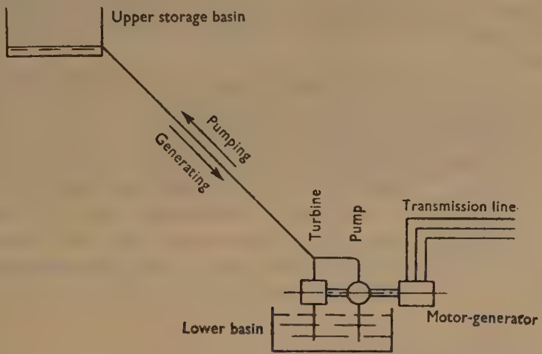


FIG. 8.—"INDEPENDENT" PUMPED-STORAGE INSTALLATION

In continental Europe during the past 30 years, where topographical conditions e often favourable, there has been steady development in the design and contruction pumped-storage plants. This progress continues. In consequence, it has been ssible to transmit energy from thermal stations in Holland to be stored by pumping ants in Switzerland. There has been less incentive in Great Britain, but projects e actively under study. One of the major ones has come to be known as the estiniog scheme; it is designed to profit by extremely advantageous topographical nditions in North Wales. These will permit a small natural lake, Llyn Stwlan, serve as the upper storage basin. By means of a dam 90 ft high and 900 ft long ilt across its outlet, the lake is to be converted into a reservoir having a maximum rface area of about 30 acres and a top water level at 1,660 ft O.D. To form the wer basin a dam 30 ft high and 1,780 ft long will be built across the valley of a small eam, the Afon Ystradau, which at present receives the natural overflow from Llyn wlan. Since the proposed top level of this lower reservoir lies at 618 ft O.D., e available working head on the system is thus rather more than 1,000 ft.

It is proposed to install hydraulic turbines and generating plant having a maximum tput of 300 MW. When working continuously for 4 hours during the peak period each day, the turbines will draw down the water level of the upper basin by about ft, and raise the level of the lower basin by about 18 ft. The pumping period, ring which the water is lifted back again, is expected to last for 6 or 7 hours nightly. is daily cycle will be followed only during 3 or 4 months of the winter season; the rest of the year the plant will be available for emergency stand-by duty, thus ing the coal which would otherwise be consumed in maintaining an equivalent acity of steam plant as "hot reserve." Allowing for stand-by operation it is

estimated that the Ffestiniog plant will generate about 300,000,000 units per annum, corresponding to a load factor of 11·4%.

Since opposition to the project is based chiefly on its threat to the amenities of the region, it should be remembered that the name Ffestiniog associated with the scheme hardly gives a just impression of the site, neither does the information that the upper dam will lie within 10 miles of the summit of Snowdon. In fact, both the tourist resort of Ffestiniog and the industrial township of Blaenau Ffestiniog are each about 2 miles from the site. The power house and the lower reservoir will lie within the small secluded valley of the Afon Ystradau, which is already overlooked by the scars of quarries and mine workings; the nearby village of Tan-y-Grisiau, which alone commands a clear view of all the elements of the project, can hardly claim to be a tourist resort.

Naturally the upper dam, at an altitude of more than 1,600 ft, is bound to be more conspicuous, but in relation to the imposing eastern face of the Moelwyn range into which it will merge, the dam will hardly be more than an incident. It is this range which will itself completely screen the whole of the works from the Snowdon massif, away to the north-west. Probably the item that will find most difficulty in shielding itself from criticism is the 275-kV transmission line by which energy will be conveyed to and from the installation. It forms the link with the national 275-kV grid now being erected by the Central Electricity Authority about 40 miles to the east. There is already an overhead transmission line within a few miles of the pumped-storage site.

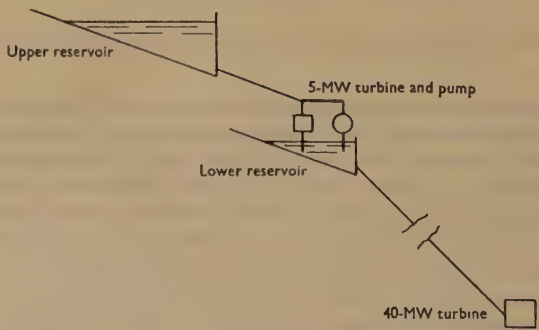


FIG. 9.—GLEN SHIRA PUMPED-STORAGE SYSTEM, SCOTLAND

Elsewhere in the United Kingdom a smaller storage scheme is already in course of operation; it forms part of the North of Scotland Hydro-Electric Board's project at Glen Shira, a few miles north of Inverary near an inlet on Loch Fyne. Like many such storage schemes it cannot be set in a precise category, but it may be said to have affinities with the systems shown in Figs 5 and 7. The main elements of the hydro-electric generating system are: (*a*) a 40-MW hydraulic turbine at sea-level; (*b*) a 5-MW turbine at 968 ft O.D.; (*c*) a small storage reservoir at about 968 ft O.D.; and (*d*) a much larger reservoir at 1,108 ft O.D. Each reservoir receives water from its own catchment, the relative sizes of the reservoirs being dictated by local topography (Fig. 9).

During normal operating conditions water from the upper reservoir will flow down to the 5-MW high-level turbine and so into the lower reservoir. Then the combined flow from the two catchments will feed the main low-level 40-MW generating set.

ut at times of heavy rainfall and high run-off the volume of water entering the wer reservoir may be greater than can be accommodated by the main turbine. nless it was safeguarded this surplus water would be spilled uselessly to waste. It with the object of minimizing this loss that the storage pump will be installed; s purpose is to lift water from the lower reservoir into the much more capacious pper one, where it can conveniently be stored.

umped storage and nuclear power

Present indications suggest that large electrical generating stations using heat om atomic fission must necessarily work under a high load factor. Because of eir high initial cost they could hardly hope to be economic otherwise. This nplies that normal variations in demand upon the transmission network cannot asily be met by changes in the output of individual atomic-energy stations. Instead, e desired flexibility of operation and the ability to meet peak loads could be chieved by interconnexion with coal-fired stations and possibly with hydraulic-orage installations. A renewed search for suitable sites for such hydraulic plants is erefore to be foreseen.

ypes of storage pump

In many instances, e.g., those illustrated in Figs 3, 4, 5, and 6, the auxiliary pumps ave no physical connexion with the main hydro-electric generating units, and there consequently no reason why they should be of special construction. Basically ey are electrically driven pumps of conventional design, with perhaps only their ceptional size to distinguish them. But in a "pure" pumped-storage installation, in Fig. 8, the pumps must to some extent adapt themselves to other elements in e station. On a single axis and perhaps on a single shaft three rotors may be ounted. They are the runner of the hydraulic turbine, the rotor of the electrical otor generator unit, and the rotor of the pump. All these must necessarily run at e same speed. During the generating period the pump rotor is idle and the ectrical unit works as a generator; during the pumping period the set runs at the me speed as before, but now the electric unit acts as a motor and the turbine runner idle. There may be mechanical clutches so that one or other of the hydraulic its can be disengaged when it is out of use, or alternatively the idle member can ntinue to rotate but it runs in air only; its casing has been emptied of water. This tter arrangement has been chosen for the 5-MW combined pump-motor-generator-rbine set at Glen Shira (Fig. 9).

In general it can be said that the diversity in site conditions that control pumped-orage installations is reflected in the variety of mechanical plant that has been eeded. Although, as a rule, the hydraulic turbine element is of the Francis type, particular instances it may be a Pelton wheel. The pump may have one stage, may have two, or even five stages. The axis of the combined revolving assembly ay be horizontal or vertical.

Recent efforts to attain greater simplicity have centred on the conception of using single machine to perform the dual duties of turbine and of pump. Since an entical principle governs the working of the two types of machine, and since in any ent a single electrical machine serves both as motor and as generator, the quest pears to be promising. A solution would be easier still if the condition of fixed tational speed could be relaxed. Because of the way in which energy losses in peline and in machine affect the operation of the plant, a given rotor should

preferably run faster when working as a pump than when working as a turbine runner under the same gross head.

An example of such a reversible pump turbine or pump-turbine unit is to be found in the Colorado–Big Thompson system already described (Fig. 6). In the Flatiron station—the lowermost station—of that system, one of the 14,000-h.p. turbines is coupled to a two-speed generator.[7] By varying the number of stator poles in circuit this electrical machine can be made to run either at 300 or at 257 r.p.m. The higher speed will be chosen when the hydraulic unit acts as a pump, and the lower speed when the set is generating. A very much larger reversible turbine set is under construction at the Hiwassie station of the Tennessee Valley Authority; its maximum rating as a turbine is 130,000 h.p.

Pumps in relation to other engineering works

The above heading lacks precision because it covers such a diversity of pumping problems. What these problems have in common is that they concern not so much the pump itself as the installation with which the pump is associated. So the overriding problem may often be, how can these other elements—civil, mechanical, or electrical—be so disposed that they give the pump the best chance of doing its work effectively.

Elements that will assuredly influence the overall efficiency of the installation are the conduits that lead water to the pump and that carry it away again. These influences may be of at least three different kinds:—

(1) If the conduits are not properly designed they may impose excessive energy losses on the water flowing through them.

(2) Conditions on the inlet side of the pump may adversely affect the flow of the water into the casing and so impair the pump performance.

(3) During the periods when the pump is being started or stopped, and when therefore the liquid columns in the conduits are experiencing rapid acceleration or retardation, violent water-hammer pressures may arise which might endanger the whole installation.

As the scale of the installation increases, it seems likely that at least the second and third of these problems will grow more acute. In fact, as this Paper has already shown, the demand for very large pumping units is growing continually. This implies that the conduits themselves may rank in their own right as civil engineering works. Almost inevitably they will be costly, yet it would be imprudent to try to reduce the cost at the risk of damaging the pump's performance.

Inlet structure

With regard to the inlet structure as a whole, a basic question is where to set the pump in relation to the lowest water surface level in the river, estuary, lake, or reservoir from which the pump is to draw. The controlling figure is the minimum absolute pressure prevailing at the pump suction flange. It is a figure which varies both with the type of pump and with the total head against which it works. It is higher for a pump of high specific speed than for a pump of low specific speed. For a given type of pump an increase in the total head implies an increase in the requisite absolute inlet pressure. The specified value is minimum or limiting in this sense, that to accept a lower value would involve the risk of cavitation in the pump. Even if the performance was not manifestly impaired, the rotor blades or even the casing might suffer gradual erosion and would in time become unserviceable.

Converting a criterion expressed in terms of absolute pressure into terms of levation above or below the free water surface gives these requirements: for a given otal head a propeller pump must be set at a lower level than an equivalent centri-ugal pump. For a given pump the elevation must be lowered as the total head ncreases. It may easily happen, then, that the pump must be situated below water evel, and that excavations must be prepared for it. Evidently the depth and the ost of these excavations can to some extent be controlled by the choice of pump; f its specific speed is reasonably low, or if its construction is such as to give the effect of low specific speed, shallow excavations may be sufficient.

Some types of pumping plant are sensitive not only to the pressure at the inlet lange, but also to the velocity distribution there. If, because of defective design of the inlet conduit—sharp corners or the like—the water entering the pump casing as a rotary or eddying motion, this whirl component may increase the power demand of the pump and possibly overload the driving motor. Axial-flow pumps of high pecific speed may in particular be affected. Additional dangers must be faced if, s often happens, the incoming water flows along an open conduit into a sump and hen ascends up a vertical suction pipe. Vortices may form on the open water urface. These will increase in intensity until they extend into the suction pipe tself, and the central core of air so entrained will pass into the pump. In extreme nstances this intrusive air may cause the pump to stop working altogether. Even f the pump manages to clear itself and to continue pumping, such periodical inter-uptions to steady flow will set up undesirable pressure surges in the piping system.

Recent studies have shown how these threats may be lightened.[8] The first essential s to see that the suction pipe is not surrounded by a wide unbroken expanse of vater surface, for such areas form breeding grounds for air-entraining vortices. The uction bellmouth should not be far from a vertical wall, which implies that the sump hould be as narrow as consistent with reasonable water velocities. Although an bvious precaution would be to submerge the pipe inlet as deeply as possible below he surface, as a defence against intrusive vortices, yet this is just the solution that vould increase civil engineering difficulties. A deep and costly sump would be equired. A compromise which allows the shallowest depth of excavation is to set he suction bellmouth quite close to the flat floor of the sump.

Pressure surges in pipes

Turning now from steady operating conditions to the abnormal conditions that ccur while the pump is being started or stopped, the most potentially dangerous nes are those arising when the power supply to an electrically driven high-head umping plant is suddenly cut off.[9] In no kind of electrical distribution system can his interruption be ruled out, and therefore the risk must be accepted as inescapable.

What will happen if, in such an eventuality, the installation is left to look after tself—the piping, it is assumed, being free of all obstructions such as valves? Quite clearly the pump rotor will rapidly slow down, it will come to rest, and then it vill begin to run backwards at a rapidly increasing speed. The water in the conduits vill likewise change its direction of motion. Yet in the end the system will stabilize tself, the pump and motor running backwards at a steady speed that is likely to be onsiderably higher than their normal forward rotational speed, and the water flowing ackwards down the pipes. If the rotating parts can withstand this treatment, the nstallation need not suffer.

Of course it is rarely practicable to allow the system to remain out of control in his way. A valve of some kind must usually be interposed in the delivery pipe,

and if this is closed at a suitably slow rate the water and the rotating machine parts can be brought to rest. What is quite indispensable, too, is that the operation should be performed automatically. Manual control of the valve must be ruled out, if only because the emergency envisaged—a power failure—may occur quite unpredictably. The design of a mechanically operated valve that has been found effective comprises a power-energized servo-motor coupled to a plug-valve or butterfly valve in the pump-delivery pipe.[10] This servo-motor comes into action immediately on failure in the electricity supply; it begins to close the valve first rapidly, and then more slowly, the motion being so adjusted that the period of reversed rotation and reversed flow in the main system is no longer than is necessary for safety.

Such servo-operated valves are included in the Granby pumping plant of the Colorado–Big Thompson project, Fig. 6. In the still larger pumps at Grand Coulee, Washington, the risk of imperfectly operated valves was judged to be so great that no control organs of any kind were inserted in the pipes. In the event of power failure a siphon outlet would limit the volume of water escaping backwards through the system; an automatic vacuum-breaker ensures that after the delivery pipe had emptied, no water from the main canal could escape.

Control systems such as those just described are intended to protect the conduit and the pump casing against excessive positive pressures, pressures that would tend to burst the pipe. There is no hope of preventing negative surges—momentary periods during pump stoppage when the internal pressure falls below normal working pressure. If this pressure drop was so severe that the local internal pressure was at any point less than atmospheric pressure, two dangers might threaten the system. First, the pipe might collapse inwards, and secondly, the water column might part and thereby give rise to excessive internal pressures when the two sections of the column re-united. This risk of internal collapse is naturally greater in a low-head system, because the delivery pipe may be several feet in diameter with pipe walls less than ½ in. thick, and it will have only negligible resistance against external pressure.

An effective safeguard, if conditions permit, is a surge tank interposed in the delivery pipe system close to the pump. Such an arrangement is illustrated in Fig. 10, which relates to an irrigation pumping plant at Goneid, in the Sudan.

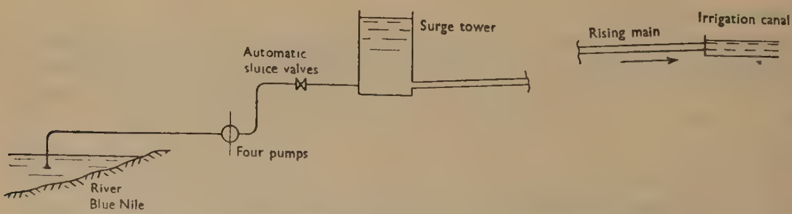


Fig. 10.—Irrigation pumping plant at Goneid, Sudan

Between the Diesel engines and the centrifugal pumps there are centrifugal clutches, so disposed that if an engine slows down for any reason the pump is automatically disengaged. Thereupon, return flow through the pipe system, and reversed rotation of the pump rotor, occur just as they do in an electrically driven set.

It had originally been proposed to use multi-flap reflux valves on the delivery mains from each of the four pumps, but an analysis showed the advantages of a single surge tower. Constructed at the intersection of the individual delivery pipes with

he single large-diameter rising main, it was both economic and effective in preventing he hydraulic gradient from falling below the pipe axis and thereby introducing the ossibility of collapse. There remained the problem of arresting the return flow own the pipe. This was accomplished by interpolating an electrically operated luice valve in the delivery pipe of each pump, arranged to close automatically at a uitable rate whenever the engine speed falls below a minimum value.

Pumps for facilitating civil engineering constructional work

Civil engineers habitually depend upon pumps of various kinds during actual onstructional operations. Even in minor works there may be a need for clearing way rainwater or infiltration water from excavations in progress. Improved types f self-priming pump are now available for such duties; they will handle air, clean ater, muddy water, or mixtures of these.

Another kind of self-priming centrifugal pumping outfit has facilitated the technique f excavation itself. In conjunction with what is known as the "well-point" system aims not merely at the removal of water that may have accumulated in the xcavated area but virtually prevents infiltration water from getting there at all. ver the whole of the site in which foundations are to be constructed, the ground-ater level is lowered so far that excavation can proceed in the dry. This result is chieved as follows. Along a circumferential line completely enclosing the area, nd at a little distance outside it, numerous small tube wells are sunk; they are all onnected to a common header pipe, which in turn is coupled to the self-priming pump-g set. When the set is started it first evacuates the header system and then raws water from the subsoil and so into the pump (Fig. 11). In time a steady

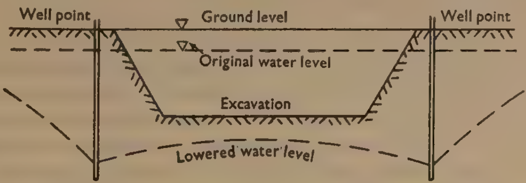


Fig. 11.—Well-point system in operation

gime is established; the ground-water level falls to the point at which the total ead imposed on the pump corresponds, as dictated by the pump characteristic, to e volume of water abstracted. Excavation can thereupon proceed without ndrance.

For continuous duties of this kind the pumping outfit will comprise separate units r handling air and for handling water, both driven by an oil-engine. In a typical stallation the piston-type dry vacuum pump will have a capacity of 60 cu. ft/min free air, whilst the 6-in. side-inlet centrifugal pump can handle 1,000 g.p.m. of ater. Together they can deal with all the water from sixty to one hundred well oints, and also with the air that may leak past joints and fittings. The tube wells well points themselves are usually 2-in. dia., spaced apart at distances of 3 ft so. They are sunk by "jetting," not driving, and in this way a natural filter of nd around each well assists the artificial gauze filter.

A successful large-scale example of the application of the well-point system was be seen during the construction of the Royal Festival Hall, London, where 340

points were served by four pumping sets.[11] In more recent works as many as eleven well-point pumps have been in use on a single site. An alternative disposition of the well points may be advantageous when trenches for large pipes are to be excavated; here, a single line instead of a ring of points will serve.

Pumps used for augmenting agricultural production

In a general way the pumps that are here considered are those used in land drainage and irrigation projects.[12]

Land-drainage problems

Taking as an example the developments that have occurred in the East Anglian fenlands, it is easy to discern a trend towards the use of water-lifting appliances compared with free-flow drainage, but this could be explained by particular local reasons. As the reclaimed peat land dried, it shrank so that the whole land surface began to sink. It is still sinking—in some areas at a rate of more than ½ in. yearly. Since drainage water could no longer escape by gravity alone to the sea it had to be raised mechanically. Another reason has intensified the demand for pumping plant. Because of the increased value of the reclaimed agricultural land, better protection from flooding was needed, not only in favourable seasons but preferably under the worst conditions of wind, tide, and weather. This multiplication of pumps, however, does not mean that they are acquiring a monopoly in handling the water. They all have to work in conjunction with existing gravitational channels. At the present moment, too, new main gravitational channels are under construction on what is by local standards an unprecedented scale.

With regard to the pumps themselves, their type and disposition show little sign of stabilized orthodoxy.[13] Particular local conditions seem to have the final word in dictating the layout of the station. Horizontal-shaft and vertical-shaft pumps, centrifugal pumps and axial-flow pumps, twin-impeller pumps, all these have been favoured. Perhaps the one characteristic common to all the pumping plants is a low load factor. They are hardly ever intended to work continuously, and indeed some of them may run for only a few days or a few weeks per year. Because, too, the lift is relatively low, rarely exceeding 10 or 15 ft, the annual cost of energy is small relative to the income derived from the crop harvested from the drained land. It may be less than 1%. This factor alone, therefore, might not have a decisive influence on the choice of motive power—whether oil or electricity. When overall costs and the question of reliability of operation have been taken into account, though, the advantages of Diesel-engine drive become clear, and that is why most of the larger English drainage stations are so equipped.

Can these overall costs be still further reduced? Small electrically driven plants need not be burdened with the wages of an attendant and with the cost of housing him; the pumps can be remotely or automatically controlled. Might it next be possible to do without the pump-house? Perhaps, in looking at this question, comparable developments in hydro-electric engineering might give guidance. Originally, both the generating sets and the switch-gear were protected by solid brick, stone, or concrete. Then, first the switch-gear, and later the power-house travelling crane, were turned out-of-doors, and they survived perfectly well. Having in this way arrived at the conception of a weatherproof pumping set, it might be compared with existing units that are not only weatherproof but waterproof too. These are submersible borehole pumps, in which both the pump and the electric

riving motor can be submerged several hundred feet below the water surface. n principle, such a set might be used for land-drainage duties; it need just be lowered ito the water.

These comparisons suggest a very simple installation comprising a vertical-shaft ropeller pump and an electric motor, weatherproof in the sense that ship's deck achinery is weatherproof. Since the piping system could be very simple, without alves of any kind, the whole outfit could be standardized and perhaps even made ransportable. Weeds and frost might be its chief enemies.

Enquiries along other lines might show that even the drainage pump itself could e eliminated. It could be replaced by an Archimedean screw. Numerous such crews, of improved construction, are now at work in the Netherlands, in successful ompetition with rotodynamic pumps.

rrigation problems

Considering a generalized irrigation project in which an alluvial plain is to be rigated with water abstracted from the river that traverses the plain, it is chiefly he scale of the project that will indicate whether free-flow irrigation or lift irrigation likely to be preferable.[14] A small scheme is best served by pumps, a large one by diversion weir or barrage across the river, and a long main canal. Since the gross ost of this diversion work will not be greatly influenced by the volume of water bstracted, it follows that the cost *per unit volume of water* will rise as the irrigated rea diminishes. A similar but less direct tendency will also affect the main irriga-on canal. On the other hand, the cost of a pumping plant drawing water directly om the river is more likely to be proportional to the area irrigated. When relatively nall areas are concerned, then, a pumped irrigation system has the advantage.

But—as foreshadowed in the Introduction to this Paper—many of the most vourable areas in alluvial plains have already been exploited. The only remaining nds suitable for agricultural development may lie 100 ft or more above river level. hey cannot be commanded by any kind of gravitational system. Pumps are bligatory, and they must lift the irrigation water against a head far greater than is istomary in an alluvial plain. Energy costs will rise proportionately, and thus a w problem poses itself: what is the limiting economical head? At what point ould the irrigation water become so costly that the crops it fertilizes would no nger be saleable?

Now in collecting evidence on this point from existing projects it soon becomes ear that many such schemes do not depend upon financial considerations alone. hey may have other aims besides the production of saleable crops and a satisfactory lance sheet. Yet their guidance need not on that account alone be set aside. ne first example depends upon pumping plant that has already been mentioned— e Goneid project in the Sudan (Fig. 10). Here the head on the pumps is 64 ft. a pumped irrigation scheme at Kom Ombo in Upper Egypt the head is a little gher; it is 76 ft. Yet since this scheme has been in successful commercial operation r more than 50 years, it can safely be deduced that the figure of 76-ft head is well thin the economic limit for these particular local conditions.

As for the very much larger Grand Coulee scheme in the United States, the rated ad on the pumps is 310 ft, but the success of this comprehensive project is not to measured in terms of crop production alone. Elsewhere, however, a head of 0 ft on the irrigation pumps has been proved feasible on a strictly commercial sis, in South Africa, for example, on the sugar plantations of the Natal Estates mpany.[15] A final example comes from Israel. Pumping schemes are now in

course of development there in which a series of pumps will impose a cumulative head of 700 ft or more, but this is another instance not meant to be assessed in terms of financial accountancy.

Sources of energy for pumping

It is illuminating to notice the sources of energy that have been chosen for the irrigation pumps just described. In the two most recent ones—Goneid and Israel—Diesel engines are installed. At Kom Ombo steam pumps were originally used, depending in part for fuel on the sugar-cane refuse (*bagasse*) that the estate itself yielded. But in later years Diesel-engine drive was found more economical. The Grand Coulee pumps are fed with low-cost off-peak electrical energy from the adjoining hydro-electric stations. Low-cost electricity is also used for the Natal Estates pumps; at some seasons of the year it is to some extent a by-product of the sugar plantations. Steam is raised in boilers supplied with sugar-cane refuse, and it serves both for process work in the refinery and for driving turbo-generating sets.

These experiences show that at present price levels oil fuel used in Diesel engines is likely to be the most convenient and economical source of energy for pumps that demand not more than about 2,000 h.p. In low-head installations of this kind, enjoying favourable conditions, the fuel consumed in raising a crop may cost less than 1% of the selling price of the produce. This figure is significant in relation to current speculations about the use of atomic energy for irrigation pumping. It suggests how unlikely it is at present that any supply company selling electrical energy generated in atomic-power stations could offer terms more attractive than those already enjoyed by users of oil engines. There is another type of competition too. When a series of irrigation pumping plants forms one item only in a comprehensive development scheme covering an entire river basin, probably storage reservoirs will be included in the project. Associated with them there will be hydro-electric stations, whose promoters may look to the irrigation pumping plants to build up their load. With regard to prospective new installations, pump owners would still have to be shown that an electrically driven installation burdened with its own transmission line would be less costly than an oil-driven plant. For the moment, then, it seems that pumps too big to be served by oil engines are the ones most likely to profit by developments in atomic-energy generation.

As for more distant possibilities, the only certainty about them is that they will all depend upon pumps. Numerous small pumps will be wanted if scattered water resources in arid and semi-arid regions are to be still further exploited by the use of atomic energy. Very big pumps relying upon atomic energy may lift water from the sea to new low-lying irrigated areas—if the cost of purifying sea-water by electrical or any other methods can be reduced far below what seems likely at the moment.

Persistent efforts to utilize solar energy for pumping have not yet had any proved commercial success. Nevertheless, although the energy of the sun cannot effectively be applied directly, it can combine with the water raised by a pump to create indirectly a valuable source of heat energy, i.e., vegetable matter that can be burned beneath a steam boiler. Examples of large-scale applications of this principle have already been mentioned, e.g., at Kom Ombo and in Natal, where sugar-cane refuse has been so used. It has also been pointed out that there has been a clearly marked trend away from locally grown vegetable fuel towards imported oil fuel. Endeavours now in progress are directed not towards large commercial undertakings of this kind, but rather towards small plants in remote areas where only local resources can be relied

pon. In the system now being perfected by the National Research Development orporation the steam plant embodies a small and simplified boiler suited to a wide ınge of natural fuel—either waste vegetable matter or a specially grown fuel crop. he prototype steam engine has two cylinders, each $2\frac{1}{4}$-in. bore $\times$ $2\frac{1}{2}$-in. stroke, hich will develop $2\frac{1}{2}$ b.h.p. when running at 1,250 r.p.m. under steam pressure of 50 lb/sq. in. If such an engine were to be used for driving an irrigation pump, it is stimated that of the total area irrigated not more than one-tenth need be set aside or growing the necessary fuel. Should such a special fuel crop be necessary, rather ıan the refuse from a food or cash crop, perhaps a fast-growing plant such as ıcalyptus wood could be recommended.

'umping from underground sources

When the water resources of a complete river system are fully developed they may ıake available a very convenient source of energy. As already pointed out this is ydro-electric energy. Being to some extent a by-product of flood-control and rigation developments it can often be offered on such attractive terms that no lternative need be considered. In India it has been used to exploit underground ater resources for irrigation purposes on quite a large scale.[16] The hydro-electric enerating stations themselves, in one such scheme, are interpolated in an existing rigation canal—the Ganges canal—and the energy is distributed by overhead lines o more than 2,000 small pumping plants. Each of these has a $12\frac{1}{2}$-h.p. electric ıotor and pump which lifts water, against a head of 10 or 20 ft, from the pervious ıbsoil in sufficient quantity to irrigate an area of about 800 acres.

Since the duty of each of the pumps in this "tube-well scheme" is $1\frac{1}{2}$ cusec, it is ıstructive to compare the project with another one that depends upon hydro-ectric energy. That is the Grand Coulee scheme. Its pumps each have a capacity f 1,350 cusecs. These figures show how wide is the range of acceptable solutions o what is a single basic problem—how to lift water for fertilizing land.

'rigation in Great Britain

Especially in the south-eastern part of Great Britain, there is a growing demand for latively small irrigation pumps. Farmers, market-gardeners, and horticulturists ı increasing numbers are finding it worthwhile to contrive artificial water supplies o make good the deficiencies in local rainfall. Since the need for water may vary om year to year and from crop to crop, fixed pumping plant may not be the most ıitable. The centrifugal pump can conveniently be mounted on a tractor, in such way that the tractor engine itself can drive it. A high-head pump would serve r spray irrigation, and a low-head pump for flow irrigation.

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The Paper, which was received on 2 September, 1955, is accompanied by eleven sheets of diagrams, from which the Figures in the text have been prepared.

Discussion

The Author, introducing the Paper with the aid of a series of lantern slides, illustrated recent progress in the development of reversible pump-turbine units. He also referred specifically to the use of various types of pumps in a newly-reclaimed desert area in Egypt.

The Chairman referred to the question of the flexibility of pumping schemes in terms of control of the amount of water being pumped. The pumps which the Author had described were capable of variation in discharge quantity by variation in speed, but it was possible to go further than that, and in the case of the axial-flow type of pump, whether single-stage or multi-stage, the variation in quantity could be controlled by varying the tip angle of the blades, even when running.

With a centrifugal type of pump, or even with the mixed-flow type, if the speed was

ried the efficiency of the pump itself would remain at its maximum value if almost the ole of the head was due to pipe friction. On the other hand, if the total head against ich the pump was operating included a large static part, then, as the speed was reduced, would run away very quickly from the maximum efficiency point. Variable speed was st satisfactory, therefore, in the case of friction systems such as booster pumps, where re was not much static lift. It also followed that the variable speed was most satis- tory when the head against which the pump had to work was reduced. With axial- w pumps, the varying of the blade angle had the effect of maintaining the efficiency at igh value for different quantities against the same total head. In other words, where e static head was very high in proportion to the total head, variable pitch axial-flow mps were suitable, again with the limitations of the head which could be generated by e axial-flow pump.

The variation of speed was possible only at some cost. When considering only electric mps, as he was doing at the moment, a reduction of speed could be achieved by losing ergy in the rotor resistance of an induction motor, and it could be achieved by using a mmutator-type a.c. motor in which most of the unused energy was returned into the ain, and where the efficiency was therefore kept at a high value. It was necessary to y, however, for that ability to control the quantity, and the more highly efficient it was e more costly the electrical plant which had to be used.

Mr W. E. Doran (Chief Engineer, Great Ouse River Board) said that centrifugal mps had been in use for rather more than a century, but it appeared that engineers did t yet know how to design a sump. Recent Papers[17, 18] presented to the Institution of echanical Engineers contained much useful information on the subject, but there was ill a great deal to be learnt.

In land-drainage pumps, a low value of the critical submergence of the suction inlet was ry important, because of the small working range on the suction side of the pump. e lowering of the sump by quite a small amount—1 ft or 18 in.—would add quite con- lerably to the cost of the installation, and the cost of lowering the sump might be more an the difference in cost between two competitive tenders.

He felt that sump design had not received sufficient consideration from civil engineers, d it was becoming increasingly important with the use of the axial-flow type of pump. y keeping the suction inlet close to the wall and close to the floor, it was possible to dis- urage the formation of a vortex. It had been suggested that the distance between the d of the pipe and the floor should be about half the diameter of the suction inlet.

He would welcome the Author's comments on the plan view of the sump. The old-

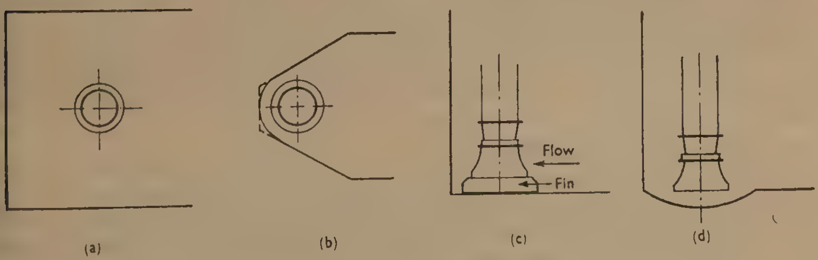


FIG. 12

shioned sump was an open-sided box, with the width corresponding very often to the idth of the inlet channel and with the suction pipe near the mid-point of one end as own in Fig. 12a. That was very liable to vortex trouble. The modern idea was to

[17] References 17–25 are given on p. 699.

make the sump rather of the type shown in Fig. 12b but to avoid expensive formwork it could be made as shown by the broken line which would probably be equally effective.

The suggestion had also been made that a diametral fin should be put underneath, as in Fig. 12c. He did not like that suggestion, because the fin might gather weeds which would affect the operation of the pump. In a situation where it seemed desirable to keep the pipe as close to the floor as possible to avoid vortex formation, it might be possible to make a bowl-shaped depression underneath the suction inlet, as shown in Fig. 12d. Did the Author think that that would be a good compromise, to avoid the expense of dropping the whole sump floor?

Putting the suction pipe too close to the wall was likely to result in uneven velocity distribution in the pipe, and there might even be an uneven load on the bearings; it was necessary therefore to strike a mean between avoiding vortices and keeping the pipe so close to the wall as to cause trouble in other ways.

In dealing with drainage pumps, the Author had mentioned the probable development of weather-proof pumps which would make a pump-house unnecessary, because the pumps could be left in the open. Mr Doran felt that any saving which could be made in that way would be extremely small, and in any case it could apply only to quite small units, because the main item of the cost of a Fen pumping station of any size was not the building but the intake flume, the delivery flume, and the delivery pipes. The cost of the building was quite a minor matter, and its provision meant that when there was an overhaul or a breakdown in bad weather there was a building in which to work. It would hardly be worth while to install weather-proof pumps and switchgear.

With regard to future trends Mr Doran thought that the most probable development in drainage pumps was towards the increased use of electric power. Electricity rates were now becoming very competitive in the off-peak periods, and the combination of a Diesel pump for use during peak hours and an electric pump with automatic control, which pumped during the off-peak hours, particularly at night, was almost ideal. With an ordinary Diesel set the natural tendency was for the pump attendant to run during the day, pumping his drains right down and letting them fill up at night. The constant fluctuation in the drain was not good from an agricultural point of view. If there was an electric set which looked after the situation automatically at night, and a Diesel set which operated during the day, it was a very good combination. There were certain direct advantages with electricity, such as the possibility of automatic control and a cheaper initial cost. The stations were usually smaller, and the cost was less provided a source of power was close to the pump. There was no need for constant attendance, and the overall maintenance costs were considerably less.

The use of electric pumps might entail the necessity for some kind of mechanical clearance on the trash racks. If the pumps were running without attendance, one could not risk having the trash racks choked with weeds, especially at night, or during the day when the attendant was not there. That was another development which Mr Doran could foresee, but he believed that electricity would be increasingly used for land-drainage pumps.

Mr H. R. Lupton (Consulting Engineer) said it had always struck him as a very clumsy and expensive method of storing energy to pump a bulky material such as water against the very feeble gravity of the earth through expensive pipes to an expensive reservoir, with losses in the pumps and pipes and further losses when the energy so bottled was used to get back heat or mechanical energy. He could not help thinking that within the next 50 years scientists would have found some more compact method of bottling energy, either chemically—as of course was done now in accumulators, when the energy was retrieved as electrical energy, not subject to Carnot's limiting laws—or, if heat was to be bottled, by the electrolysis of water. There must be, he thought, already existing methods of bottling energy very much less clumsily than by pumping water to an expensive reservoir. Very soon, no doubt, it would be possible to store the energy inter-atomically, when the bulk of the store would be very much smaller.

Looking still further ahead, he could not help thinking that Malthus's pessimistic ·ognostications were almost certain to be overcome by scientists, possibly in the near ture. It seemed the reverse of wisdom to use a great deal of energy in irrigating areas here nature could not at present grow vegetation when that energy might be used for the rect synthesis of the products which were needed—a very much more direct method. hat was already being done, for instance, with rubber, and he had no doubt that in the ext 50 years scientists would develop other applications.

The Author, in his book "Land, Water and Food," had stated that it was estimated that the increase in food needed by the world in the next 50 years, probably not more than % could be provided by large-scale engineering schemes of irrigation. That was a poor ospect, and emphasized Mr Lupton's contention that food, clothes, and other requireents for civilization should, in the course of the next 50 years, be increasingly provided ot by irrigating land to help nature but, still using nature wherever nature worked ficiently, by synthesizing the products instead of bolstering up nature to grow them.

Mr Lupton regretted that the Author had not dealt more fully with the great increase in itomatic working in pumping plants. It was particularly applicable, of course, to small stallations; for instance, in the case of water supply, to small re-pumping stations, and ill more, in the case of sewage plant, to small sewage stations, because generally the plant as of low horsepower, and the cost of attendance formed a very large proportion of the hole cost of the service. It was where that occurred that automaticity should be creased at an accelerating rate. In industry, of course, there were innumerable instances automatic devices controlling processes of great variety.

The electronic control of water supply was no new thing, and it should be possible to velop electronic devices to a greater extent.

Coming to a small point of detail, he noticed that in Fig. 4 the delivery pipe from a imp ended above the water level. That was a great sin; that pipe should certainly dip to the water to get the benefit of siphon action, and he was surprised that it should have en shown otherwise. If it was desired to stop the water siphoning back when pumping as not taking place, a siphon-breaking air-valve could be arranged at the top of the phon.

Dr D. F. Denny (Research Engineer, British Hydromechanics Research Association) served that the hydraulic design of inlet structures was of primary importance to the ccessful functioning of the whole system. Particularly with the very large schemes hich were in progress or envisaged for the future, it was es ential that the size of the take structure and the depth of excavation be kept as small as possible. But at the same ne it was necessary to ensure that that flow in the intake did not have an adverse effect the performance of the pump. Large pumps could now be designed with efficiencies of e order of 90%, but in the case of high-specific-speed pumps, that performance very often pended on good flow conditions on the suction side.

To the sources of swirl in suction pipes listed by the Author, Dr Denny would add the mp itself; because, if the water swirled in the sump, that swirling motion persisted as e water flowed into the pump, and the effect was either to reduce the delivery of the mp or to overload the driving motor. Sometimes the swirl in the sump was localized o a vortex, the tail of which reached the suction pipe and allowed water to be drawn in. st pumps did not like air, and experiments showed that 1% of air entering with the ter could reduce the efficiency by as much as 10–15%. That decrease in efficiency could ss unnoticed if there was no means of measuring the flow through the pump. The one re way to avoid swirl and vortices was to make the sump very deep, but that was a very pensive remedy. However, by careful attention to the shaping of the boundary walls was possible to control the flow patterns even in shallow water.

Recent research had led to the framing of guiding rules for the design of sumps, but it s still difficult to predict the performance of individual sumps. For that reason it had come more usual, particularly with large schemes, to investigate the flow conditions the sump by means of a scale model. Such models could provide fairly accurate

quantitative data for the design of an inexpensive sump which at the same time would not adversely affect the performance of the pump.

Dr Denny then showed an illustration of a 48-in.-dia. pump inlet submerged in a power-station cooling-water sump. A 10–15-in.-dia. vortex had formed in the sump and the pump ran very roughly. He then illustrated a $\frac{1}{12}$-scale model which almost exactly duplicated (to scale) the vortex formation and flow patterns. The model had been used to find means of avoiding the swirl and the vortex. The erection of steel baffle-plates in the sump had prevented the vortices both in the model and in the full-scale installation.

Fig. 13 showed an unusual view of a vortex, taken through the glass side of the tank. The vortex, which appeared as a trumpet on the surface, had an air core stretching down

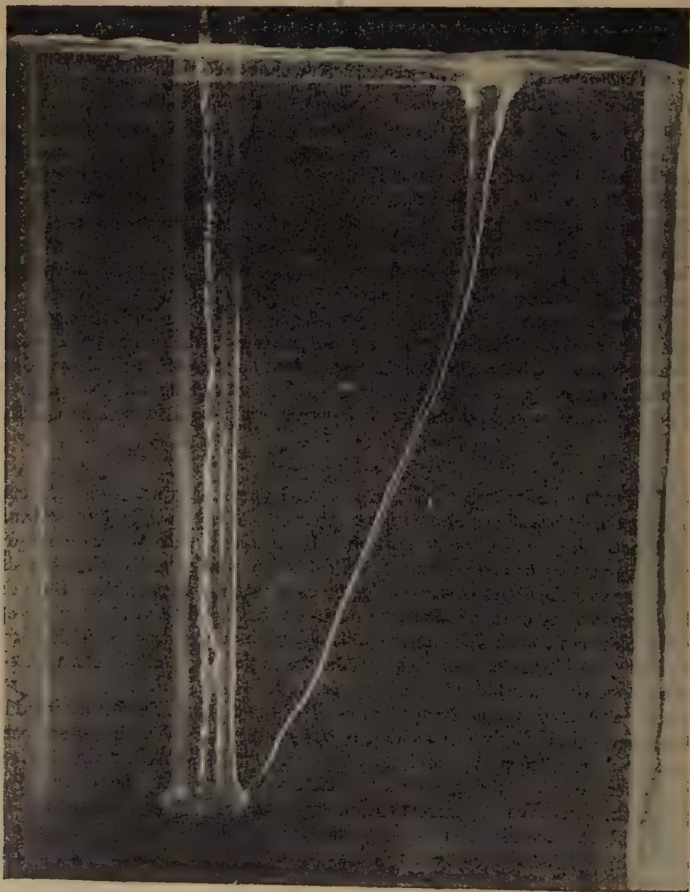


FIG. 13

to the bell-mouth, and air was entering the pipe. It illustrated how deep the vortex core could go if the flow conditions in the sump were bad.

Mr Gerald Lacey (Consultant, Drainage and Irrigation Adviser, Colonial Office) said that the Author had listed, amongst the various uses to which pumps could be put, the augmenting of agricultural production. So far as irrigation was concerned, engineers

:e faced nowadays with the larger problem of integrating land, water, and power
ources. Of those power resources, a part could be devoted to pumping, either for
inage or for irrigation, and both were very frequently required.

The Author had referred to the exploitation of underground water resources for irriga-
n in India and Pakistan. That had been done in the United Provinces, and the first
ge project of that kind had been due to the vision and energy of the late Sir William
mpe, under whom Mr Lacey had had the privilege to serve. Twenty years ago there
l been no fewer than 1,500 of the tube wells to which the Author had referred in the
ited Provinces, and all those wells had been operated by power derived from falls on
Ganges canal. It went further than that, however, because in addition to those falls
the canal there had also been thermal stations. There had been a grid extending over
reat part of the province, and that grid also supplied power to pump water from the
mgunga river to the Ramgunga canal. In fact the first base load had been the
mgunga canal pumping station and the large-scale development of tube wells had fol-
ved. That grid was still expanding, and it was an early example of the integration of
d, water, and power resources.

In many countries water was pumped from rivers for irrigation, the Nile in Egypt and
Sudan, and the Tigris in Iraq being notable examples. In the near future there would
all probability be two or three new major canal systems constructed in Iraq. When
ge sums were to be spent on projects of that character, and when there was soil of which
full productivity had not been tested, it was always advisable when possible to have a
ot scheme to try out in advance and on a small scale what it was hoped to achieve when
gravity canal works were ultimately constructed. For that purpose, a pumped pilot
igation project from the nearest river was invaluable. As the Author had pointed out,
ere were many cases where pumping was decidedly more economic than a gravity system,
rticularly for small works, or when a gravity canal involved a long haul for the water
er difficult country.

One thing which had not been emphasized in the Paper was the extent to which pumping
rough pipes should be cut down whenever possible. If the levels permitted, it was much
tter to pump in a number of stages, giving the effect of a staircase with open channels
tween the steps, than to have a long delivery pipe, because a great deal of head was lost
the latter process. During the war Mr Lacey had been concerned with a water supply
stem for Jodhpur, where water was brought from a range of hills some 90 miles away,
–90 cusecs of water being taken across country in a lined open channel, siphoned under
umber of rivers and finally pumped up in step-by-step fashion into a storage reservoir.
e method had been adopted during the 1939–45 war when pipes had been unobtainable;
had proved to be cheaper and more economical in power.

The Author had referred to Diesel engines as being very useful for small pumping
emes. Mr Lacey suggested that so far as possible a Diesel-electric system should be
ed, with a Diesel-engined central power house and pumping stations tapping a small local
id.

Iraq was very fortunate in having two sources of power—oil and water. There were no
ge falls on the canals, but large storage works now under construction would provide
wer, and in any future overall power development a compromise between oil and water
d to be made.

Mr M. R. James (Planning Engineer, Metropolitan Water Board) quoted from p. 666
d said that to the words "right size of pump, the right number of pumps, and the right
ce to put them" he would like to add "the right types of pump".

Every drop of water used in London had to be pumped at least once—most of it twice
d some more often. The extent and complexity of the distribution system made it
sential to ensure that the right types of plant were employed and that head losses were
luced to the minimum.

Pumping in London was not confined to distribution and Mr James described with the
e aid of a wall diagram (reproduced as Fig. 14) an installation at the Hampton works of

the Metropolitan Water Board for pumping raw water to storage reservoirs by utilizing available excess head.

It was the normal practice for all water which had to be purified to receive its first stage of purification by passing through open storage reservoirs, which had been built primarily for the purpose of storing water against drought. In its passage through those reservoirs it underwent sedimentation, equalization of quality, and the reduction of pathogenic bacteria.

A substantial proportion of the water which was purified at the Hampton works came from an intake on the River Thames about 1½ mile above the works, at Walton, where it was pumped into a couple of storage reservoirs with a top water level at 65 O.D. and then flowed by gravity down to the inlet of the rapid filter plant at Hampton, which was at a level of 33 O.D.

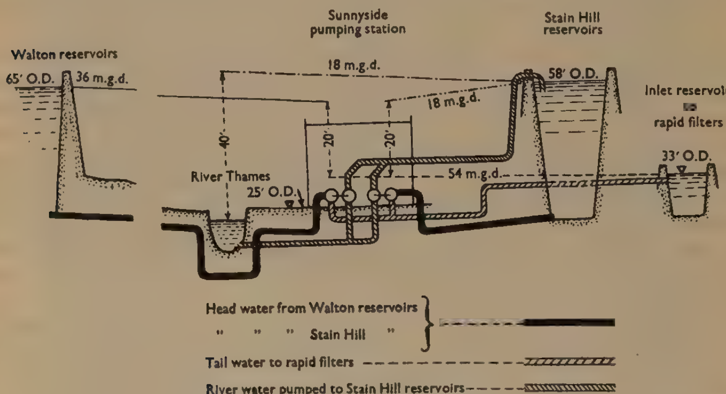


Fig. 14.—Hampton works. Sunnyside pumping station

There were also at Hampton two much smaller reservoirs (known as the Stain Hill reservoirs) with a top water level of 58 O.D., which could also discharge to the rapid filters at 33 O.D. In both cases there was about 20 ft of excess head. Adjacent to the intake was a small pumping station with five rather comprehensive units, each consisting of a water turbine, an electrical machine which could be used either as a motor or as a generator and a pump. The suction of each pump was connected to the intake, and its delivery to the inlet main of the Stain Hill reservoirs. The overall efficiency of the turbine-driven pumping sets was approximately 66%.

Assuming that 18 m.g.d. was allowed to flow by gravity from the Stain Hill reservoirs through one of the water turbines to the filtration plant, that quantity would pump 6 m.g.d. from the river to the reservoirs. That meant that 18 m.g.d. had come out and only 6 m.g.d. had gone in, and so in a few days the reservoirs would be empty and they would have lost all their head unless something else was done. To make up the difference of 12 m.g.d. 36 m.g.d. flowing from the Walton reservoirs to the filters at Hampton was made to pass through two more water turbines, which operated two 6-m.g.d. pumps to lift the balance of 12 m.g.d. to the Stain Hill reservoirs.

Reduced to its simplest terms, the installation enabled 18 m.g.d. to be lifted from the Thames at Hampton for circulation through the Stain Hill reservoirs and saved the fuel which would otherwise be necessary to lift an equal quantity of water from the intake at Walton into the Walton reservoirs. With present-day prices of coal and electrical energy—the plant at Walton was either steam or electrically driven—the annual saving in energy, if the plant were kept in operation throughout the year, would be about £12,000

ne station ran unattended except for a cleaner during the day. The electrical machine each set had two main purposes. The first was to assist the water turbine to work the mp should the water level of the reservoirs fall below the normal, and eventually to take er the whole of the pumping from the turbine if the reservoirs had to be completely pleted. The second was to make use of a larger amount of excess head which would come available when a new reservoir was built, close to the existing Walton reservoirs t farther south, having a higher top water level. The turbines would not only do the mping but also generate current which would be fed to the bus-bars of the main pumping ation.

The electrical machines had hollow shafts and three clutches, which enabled any of the lowing combinations to be used:

The turbine to drive the pump with the electrical machine idle.
The turbine and the electrical machine to drive the pump together.
The turbine to drive the pump and also the electrical machine as a generator to supply current to the works.
The turbine to drive the electrical machine as a generator, without the pump.
The electrical machine to drive the pump with the turbine idle.

The plant had been installed in order to make use of excess head which existed to satisfy ner requirements. It would not have been advisable for the Board to have the reservoirs signed with their present top water levels solely to provide pumped storage with all the advantages enumerated by Mr Lupton.

Mr G. A. Bonnyman (Sir Alexander Gibb and Partners, Consulting Engineers), erring to the irrigation scheme at Goneid illustrated in Fig. 10 of the Paper, said that e scheme had been installed to pump water from the Blue Nile up to the distribution annels at a higher level, and it was the largest irrigation scheme in the Sudan.

At Goneid a large proportion of the water for irrigation was required when the river was flood; however, when the river was at a low level a small quantity of water was also quired, so that there was the problem of ensuring that the pump suction would work isfactorily under those conditions. The Author had stated that the first essential was ensure that the suction was not surrounded by a wide, unbroken expanse of water face. At the station in question, the location of the intakes sufficiently far out in the er to obtain adequate submergence at low Nile, resulted in just the condition to which e Author referred. The presence of silt and sand had ruled out a number of alterna- es. Accordingly, after a model test it had been decided to arrange a wall close to the ck of the intake pipes. The screen installed in front should also have a beneficial effect. e present season was the first for which it had been in operation, and it was working te satisfactorily. There was only another month to go before the lowest river level s reached, and then all would be well.

The general layout was very favourable to the installation of a surge tower. In order t to exceed the maximum permissible suction head for the pumps, it had been necessary install them about 22 ft below ground level, and therefore the height to the top of the ver above ground level was relatively small; it had been possible in that way to provide inexpensive solution of the problems of surge and water-hammer.

As was well known, the axial-flow pump was suitable for low heads and the centrifugal mp for higher heads. What did the Author consider to be the most suitable change- r point from the one type to the other?

Mr M. W. Leonard (Soil Mechanics Ltd) used a series of slides to illustrate various thods of ground-water lowering in general use by civil engineering contractors.

Mr K. H. Tuson (Partner, Messrs Mackness and Shipley, Consulting Engineers) erred to the difficulties which appeared to exist in the design of a combined pump and bine for pumped-storage schemes. Could the Author say why the design had not gressed further than it seemed to have done? In view of the amount which would be

saved, on machinery, building, and excavation and of the complication of three clutches and so on, it seemed that it was a study which mechanical engineers might find it well worthwhile to pursue with vigour.

Mr I. C. Forbes (Engineer, Dorchester R.D.C.) put two questions to the Author. Some time ago he had been interested in a water-supply scheme, where the top water level of the distribution reservoir was 786·00 O.D. The length of the rising main was 4¼ miles, and the standing water level in the borehole was 220·00 O.D. with the station floor level 256·00 O.D. (Fig. 15). The station had turned over from Diesel-engine drive to

FIG. 15

electrical power, and there had been trouble at once when the current failed. The sudden cessation in flow had caused a pressure drop in the main, and the pressure wave, whose initial point was at the impeller tip, had travelled the whole length of the rising main, with a velocity of approximately 4,600 ft/sec. It had been found by calculation that the time lag between the times when the pump cut out and the back surge from the reservoir occurred was 4½ sec, and the conclusion had been reached that no surge suppressor would be able to act quickly enough to prevent damage. On those grounds it had been decided not to install a surge suppressor. Would the servo-mechanisms mentioned by the Author act sufficiently quickly to overcome that surge and resurge? Mr Forbes then referred to the Humphrey gas pump which had been popular about the year 1910. It had struck him as being a very capable pump for handling large volumes of water, but since it had been installed in a reservoir for the Metropolitan Water Board, he had never heard much about it, and wondered whether or not it had fulfilled what had been claimed for it at the time.

Mr M. R. James, replying to Mr Forbes's question about the Humphrey pumps at Chingford, said that they had been quite successful in the work for which they were designed; their load factor had been low, because their duty was to refill the King George Reservoir whenever it had to be used to supplement the flow of the Lea so as to maintain the supply to the Lea Valley works. Periods of drought occurred relatively infrequently and the result had been that the Humphrey pumps had not worked a great deal. Their load factor over the past 43 years had been only about 2½%.

*** **Mr P. Dériaz** (Chief Designer, Water Turbine Department, English Electric Co. Ltd) referred to the Author's mention of dual-duty runners, capable of running as turbines and as pumps. Examples of such reversible pump turbines were the Colorado–Big Thompson, the Flatiron, and the Hiwasse pump turbines. Reversible pump turbines with movable blades of the Kaplan type had also been considered for very low heads (especially in France for tidal schemes).

The origin of the mixed-flow variable-pitch reversible pump turbine, now being installed at Niagara Falls was worth mentioning. Early in 1952, it had been noticed that some of

*** This and the following contribution were received after the closure of the oral discussion.—SEC.

ne largest Francis turbines were being run continuously at very low gate opening with onsequent wastage of water. The possibility of disturbances on the extensive distribution network had made it necessary to provide for emergency power supply at the very hortest notice and that was the reason which had been put forward by the operating ngineer for such unorthodox procedure. To the designer of the turbines, that state of ffairs was very disappointing. To remedy it would require a turbine having its maximum efficiency at small gate opening instead of the usual seven-eighths of full rated load or a Francis turbine of medium specific speed. Reference to Fig. 16 would show that the Kaplan turbine approached that requirement. On the other hand, the Francis turbine f medium specific speed (n_s = 216(metric)) gave, at 0·4 of rated power, an efficiency points below the Kaplan of n_s = 420. The shaded area illustrated the importance of he improvement required. The diagram also showed why the Kaplan turbine had

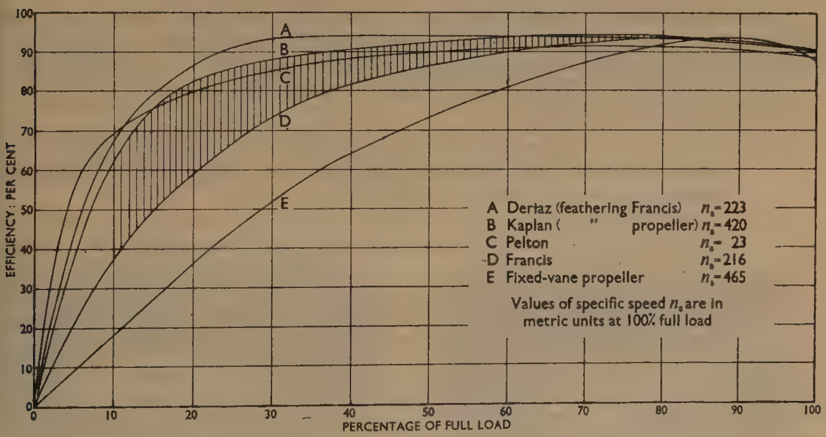


FIG. 16.—COMPARISON OF RUNNER EFFICIENCIES

usted the propeller-type fixed-blade turbine in so many cases. Now it was particularly he Francis turbine, associated with storage, short pipelines, and heads ranging from 00 to 700 ft, which was interesting.

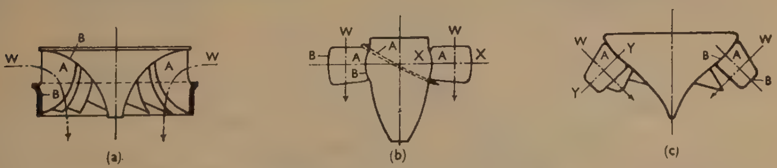


FIG. 17.—COMPARISON OF FRANCIS, KAPLAN, AND DÉRIAZ TURBINE RUNNERS

(a) Francis (inception about 1870, Swain)
. Fixed vanes
. Toroidal surfaces of hydraulic profile
/. Direction of flow: radially inwards, axially outwards

(b) Kaplan (inception 1913)
A. Adjustable vanes about axis XX
B. Spherical surfaces of hydraulic profile
W. Axial direction of flow

(c) Dériaz (1952)
A. Adjustable vanes about oblique axis YY
B. Spherical surfaces of hydraulic profile
W. Oblique direction of flow

In 1952 Mr Dériaz had set himself the task of investigating the possibilities of applying the principle of the Kaplan turbine with its feathering vanes to a head of 300 ft, i.e. exceeding anything which had been attempted so far. The Kaplan turbine had so far been developed to a maximum head of 200 ft. Attempts to exceed that head had encountered very serious difficulties:—

1. A lower specific speed was necessary than that of a normal Kaplan turbine because of cavitation conditions and efficiency. The hydraulic requirements of a lower specific speed was the short expanse in a radial direction of the inlet edge of the turbine runner vanes. Compared with a Kaplan runner, there had to be more vanes and an increased ratio of vane length to pitch. The inevitable result was the mixed-flow layout, Fig. 17c.
2. Mechanically the enormous loads to be supported by the runner-vane pivots required a very large hub diameter, to locate large bearings.

It became obvious for all those reasons that the axial-flow character of the Kaplan turbines (Fig. 17b) had to be abandoned and that the mixed-flow position of the Francis runner vane (Fig. 17a) associated with the feathering possibility for altering the pitch, provided the solution. The new turbine thus conceived was shown in Fig. 17c. Its principal features were as follows:—

1. The oblique "mixed flow" position of the runner-vane trunnions, with the advantage of the short radial expanse in the vane inlet edge, associated with larger distances between the bearings of the vane trunnions.
2. The spherical surfaces of the hub and skirt to permit rotation of the vanes whilst maintaining close clearances. (A departure from the corresponding toroidal surfaces of the Francis runner.)
3. The runner vanes were of simpler shape than those of the Kaplan and could be made to close over their entire length, where they met each other in the closed position, where the runner could act as an effective shut-off valve.

A turbine designed on those principles and tested gave remarkable results, the efficiency curve showing an improvement even on the Kaplan turbine (Fig. 16). There was no doubt that such a turbine had many striking advantages over the Francis which it would replace in many cases as the Kaplan had replaced the fixed-vane propeller runner.

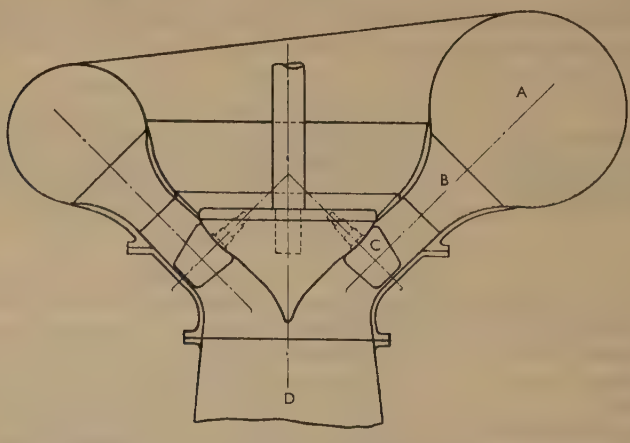


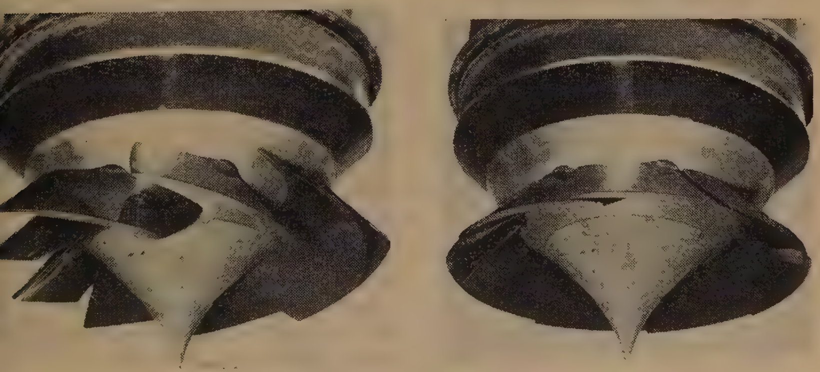
Fig. 18.—Revised form of spiral casing and diffuser
A. Spiral casing, oblique design. B. Fixed diffuser blades. C. Variable-pitch runner vanes. D. Suction pipe

In the spring of 1953 The English Electric Company had decided to enter the new field reversible pump-turbines and develop machinery for the hydraulic conditions set out the Hydro-Electric Power Commission of Ontario for their pumped-storage scheme Sir Adam Beck No. 2 generating station at Niagara. In view of the large fluctuations head of that pumped storage, a feathering machine offered some definite advantage er the fixed-vane conventional pump-turbine because of its flexibility and it had been cided to go ahead on the design of an entirely new runner combining the characteristic a reversible pump-turbine with the flexibility given by movable vanes.

In view of the very large size of the motor-generator, synchronous machines were the le. They had a relatively low starting torque and might be difficult to bring to syn-ronous speed owing to the heavy torque required by a conventional pump, even with ut gate. That difficulty was obviated with the conventional pump by depressing the iter level in the pump-suction pipe so as to permit starting the pump with the runner ear of the water. With the feathering machine that difficulty disappeared because the rque required to drive the turbine runner in shut position was a very small fraction of e normal torque. Provision for compressed air with its bulky tanks and compressors came unnecessary. The start would take place very much more quickly.

The reversible version of the new turbine had obviously to take into account the re-irements of high efficiency when working as a pump as well as a turbine. To that ect a completely revised form of spiral casing and diffuser had been developed (Fig. 18).

The runner is shown in Figs 19a and b.



(a) VANES OPEN (b) VANES SHUT

FIG. 19.—DÉRIAZ RUNNER

Preliminary tests carried out on a 24-in. model under 12-ft head showed excellent omise. It had then been decided, at the suggestion of the Hydro-Electric Power ommission of Ontario, to repeat the tests under full prototype head, ranging from 90 to) ft. A model turbine pump of a smaller size had been used.

From tests under prototype head, it had become clear that full head could be developed the feathering pump at any opening of the runner vanes. No instability had been served. That was of considerable importance, since the starting operation of the pump ould be carried out smoothly and the power input controlled to any desired load.

Whether running as turbine or as pump, the runner could be made to run at optimum iciency. That was of great importance, especially for the pump which was efficient at ll head also.

As a result of those tests the Hydro-Electric Power Commission of Ontario had ordered x units of 52,500 b.h.p. each for their Niagara pump-turbine station.

Dr Charles Jaeger (Consulting Engineer, English Electric Co. Ltd) believed that in the near future pumped-storage schemes would acquire considerable importance. His opinion was based on the probable increase in the cost of coal at a faster rate than that of the average cost of living, and even more so on the future development of nuclear power and new technical developments in pumped-storage schemes.

In spite of pessimistic basic assumptions a price estimate by Haldane and Blackstone[19] showed that pumped-storage schemes were a paying proposition for the conditions prevailing in Great Britain. For conditions obtaining in a power economy based mainly on water power, price estimates given by Musil[20] led to similár favourable conclusions.

Very little information was available on the probable structure of prices for nuclear energy. It was assumed that capital costs would be high and that that type of energy would be base load. Most experts were of the opinion that even in countries now widely depending on steam power the production of additional peak-load energy would be by hydro-power of the more flexible type and/or by pumped-storage power.

When comparing thermal peak power and pumped-storage power,[21] the possible improvements in the overall efficiencies in the near future for both types of schemes should be considered. It was to be expected that thermal-power efficiency would be steadily increasing at a slow rate. The overall efficiency of a pumped storage scheme was given by the formula:

$$\rho_1 = \frac{H_T Q_T T_T}{H_P Q_P T_P} \eta_T \eta_P$$

where H = gross head,
Q = maximum discharge,
T = time during which station runs loaded at full capacity,
$QT = V$ = the total volume of water passing through the runner,
η = an overall efficiency factor including pipe losses.
The subscripts P referred to pumps and T to turbines.

True pumped storage corresponded to the conditions: $V_P = V_T$, $H_P \simeq H_T$, and therefore $\rho_1 = \rho_1{}^* = \eta_T \eta_P$ (as in the Ffestiniog project in Wales).

For true pumped storage a value of $\rho_1{}^* = 0{\cdot}6 - 0{\cdot}70$ was often assumed, neglecting the friction losses in the pipes, which should be kept as low as possible for that type of station.

Mixed pumped storage corresponded to the case where $H_T > H_P$ and/or $V_T > V_P$. The case $H_T > H_P$ was illustrated by the Sron Mor pumping station[22] (Scotland) where the water pumped from the lower reservoir up to the higher larger reservoir flowed back and through the turbines and also through the turbines of Clachan power station (Fig. 9). For Sron Mor, $H_P = 138$ ft and $H_T = 138 + 956 = 1{,}094$ ft. The Niagara Falls pumped-storage scheme also worked economically with $H_T = 292$ ft and $H_P = 90$ ft.

That definition of ρ_1 showed how, given favourable natural conditions, the overall efficiency could be improved.

If $$\rho_2 = \frac{P_{\text{peak}}}{P_{\text{off}}} = \frac{\text{Value of on-peak energy in pence per kWh}}{\text{Value of off-peak energy in pence per kWh}}$$

the P_{peak} value denoting the value of power (usually given in £ per kW) expressed in pence per kWh, then the simplified expression $\rho_2\rho_1 \geqslant 1$ represented the conditions for a pumped-storage scheme to be economical, provided that capital charges and operation costs were being neglected. (A value of ρ_2 from 2 to 3 could be assumed for normal present-day conditions but higher values could be foreseen when nuclear energy developed.) The condition $\rho_2\rho_1 \geqslant 1$ would approximately apply to the hypothetical case when a pump-turbine could be installed in an existing hydro-power station, where dam, reservoir, tunnels, and pipelines already existed, and were paid off by running the station on conventional lines without pumping.

More generally:

$$\rho_2{}^* = \frac{P_{\text{peak}} - P^*_{\text{charges}}}{P_{\text{off}}} = \frac{\text{Value of on-peak energy} - (\text{capital charges} + \text{operation charges})}{\text{Value of off-peak energy}}$$

and the more general conditions for a true or a mixed pumped-storage scheme to be economical were:

$$\rho_2^*\rho_1^* \geqslant 1 \text{ or } \rho_2^*\rho_1 \geqslant 1$$

Capital charges, operation charges, and all types of losses were included in ρ_2^* and ρ_1.

Turning to modern trends in pumped-storage design, the average turbine efficiency η_T, under varying conditions of head, could be vastly improved if a Dériaz variable-blade impulse turbine[23] was substituted for a conventional Francis turbine. It had been shown that the efficiency curve of a Dériaz turbine was flatter than that of the Kaplan turbine and its overall efficiency was high when compared with the accepted values for a conventional Francis runner.

A similar observation applied to the Dériaz pump runner, giving high average η_P values.

Finally, under special conditions a Dériaz reversible pump-turbine with movable blades could be considered.

A reversible pump-turbine had been developed for heads ranging from 43 ft to 90 ft for the Niagara pumped storage station. The Dériaz turbine had been designed for heads

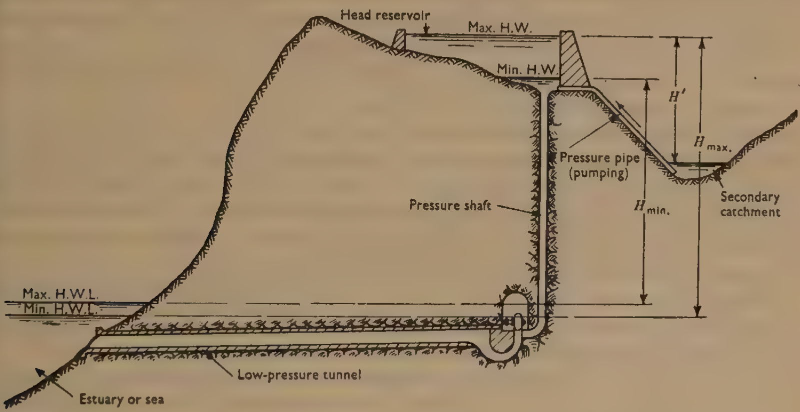


Fig. 20.—Possible applications, to a coastal ridge of hills, of a pumped-storage scheme with underground power station

of up to 300–350 ft, and higher heads were being considered. It had been objected that —for the time being—no design existed for a wider range of heads; that objection lost some of its strength when it was considered that the range which was already covered by existing designs was by far the most important for pumping purposes. Many large rivers were dammed by 300 to 350-ft-high dams where pumping stations with short pipes (reduced cost and reduced friction losses) could be established.

Whenever reservoirs were to be built in the near future, the possibility of further pumped storage to be added later should be borne in mind at an early stage. Tunnels, pipes, and surge tanks had to be built accordingly.

The advantages of underground power station designs for pumped-storage schemes should be closely investigated. Fig. 20 showed some possible applications to a coastal ridge of hills.

Pumped-storage developments would require the most economical designs to be adopted: very light buttress dams, prestressed concrete dams, etc., would have to be considered for daily or weekly reservoirs for peak-energy prices to be kept as low as possible.

It was commonly argued that most of the resources of new hydro-power still available on the continent of Europe would be exhausted within the next 15 or 20 years.[24] Before that, a new trend would be set into motion and existing conventional hydro-power schemes would be turned into mixed-storage schemes.

The Author, in reply, said that Mr Lupton made a number of very interesting points, and in particular had commented on the automatic working of pumping plants. In a Paper of rigidly defined length it had been necessary to omit many such topics. With regard to the absence of siphonic action in the Tarraleah plant, Fig. 4 had been taken from the constructional drawings supplied by Mr Knight, of the Tasmania Hydro-Electric Commission, for whose energy and ability the Author had the greatest respect. He had seen the plant, and he had no doubt that the siphon pipe had been omitted because it was a temporary installation, and it might have been considered better to leave the plant as it stood rather than to use a reflux valve or something of that sort. The question of the submerged delivery pipe had often caused comment. On one occasion in Egypt a salesman had tried to convince a landowner of the advantage of a submerged delivery pipe as against allowing the irrigation water to splash many feet below the pump outlet, whereupon a rival salesman had tried to sway the landowner the other way by saying "Pasha, what do you feel when you put your hand over your mouth?" "I feel suffocated," said the Pasha. "So does the pump," replied the salesman.

Dr Denny had given an admirable exposition of his work, which made very clear the necessity for careful study in designing sumps.

Mr Lacey had spoken of the use of canals instead of pipes. Very often a combination of the two was used. Mr Bonnyman could explain that in the Goneid scheme there was a relatively short delivery pipe and the main canal began as soon as the necessary height had been reached. Mr Lacey had spoken of competition between Diesel and electric plant, and there was a fascinating range of opinion on that subject. Mr Doran had also referred to that matter. By avoiding the peak-load periods it might be possible to use electricity where before it had not been competitive.

Mr James had described a very interesting project. It was surprising to learn that in the Metropolitan area there was a pumped-storage scheme actually at work. It showed the great attention that the Metropolitan Water Board paid to making the most effective use of water power and not wasting energy if they could possibly help it.

Mr Tuson had spoken of the difficulty in pumped-storage plants of using the same machine both as pump and as turbine. The Author thought that the difficulty did not lie in actually using a single machine, but in getting it to work at the highest possible efficiency; because storage pumps must work at a high efficiency all the time if the scheme was to be economically sound. Comparing a given rotor working first as a pump and then as a turbine, at a given speed and discharge, it could be shown that the head generated by the pump would be less than the head applied to the turbine. That was because of the reversal of the direction in which the internal energy losses operated. There was a corresponding difference in the direction of the external energy losses, e.g., friction losses in the conduits between the upper and lower reservoirs. Nevertheless, if regard were paid to the normal head-discharge characteristic curve of the pump, it was true that at a *reduced* discharge it might be possible to generate the desired head, but because of the shape of the efficiency curve, the machine would be working at a much lower efficiency. That was the basic difficulty of getting the same constant-speed machine to work as pump and as turbine. That was why in one of the schemes mentioned in the Paper there was a two-speed motor-generator, which worked at a high speed when pumping and at a lower speed when generating.

With regard to the Ffestiniog pumped-storage scheme, Mr Tuson had pointed out that its development would interfere with the Ffestiniog railway. In reply, the Author said that when he had made his reconnaissance in 1955 to see the site, he had been duly sentimentally affected by the deserted railway, but he would point out that with existing schemes in Scotland there had been railway lines which had had to be rebuilt at a higher level. At Loch Treig on the Lochaber scheme that had had to be done, and if there were

ufficient enthusiasts to support the Ffestiniog Railway he thought that it could be done here also.

Mr Forbes had asked whether a surge suppressor could be designed to operate in a very mall period such as 4½ sec. The Author thought that it could, having regard to the very owerful servo-motors used on American schemes.

In reply to Mr Doran's questions about suction sumps, the Author thought that either of he types represented in Figs 12b and d would be advantageous. Where multiple pumps vere concerned, information had recently been presented in American Papers.[25, 26] As or the possibility of installing mechanical self-cleaning trash-racks in drainage pumping lants, perhaps advantage could be taken of the performance of similar but larger self-leaning devices in sewage works, circulating-water pumping systems in steam power-tations, and in water-power plants. There might be difficulties in designing an apparatus hat would be sufficiently simple and inexpensive for the proposed new duties.

Mr Lupton's advocacy of direct synthesis as opposed to pumped irrigation for the roduction of food and clothing raised questions that could hardly be answered in a few aragraphs. Quite often, the object of a land-reclamation scheme was not merely to row food: it was to furnish a livelihood. As world population expanded, more and more housands of people would want new lands where they could build towns and villages for hemselves, and fields where they could work. Even if energy had to be expended in umping water on to those lands, it made available vastly greater supplies of energy, that vas, the energy of the sun without which nothing would grow. Moreover, an indispens-ble prerequisite to any process of food synthesis was a supply of raw material of various inds, ready to be synthesized. If those materials were mineral in origin—say oil or oal—they would eventually be exhausted as mines or oil deposits were worked out. If egetable matter was needed, then the search for that would take us back to the starting oint—the land.

Mr Bonnyman had asked about the most suitable head which marked off the operating one of axial-flow pumps from that of centrifugal pumps. Although there was no rigid imit, a figure of 20 ft head might tentatively be offered. In particular circumstances the hoice of pump might be guided by other considerations than the total head, e.g., the naximum suction lift might be decisive.

Mr Forbes had referred to the Humphrey internal-combustion pump, and Mr James had nentioned the Chingford installation. Similar pumps had been used for lifting irrigation vater from the Murray River in Australia; but perhaps one factor that restricted their ield of action was their preference for a fairly constant suction-water level and total lift.

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Correspondence on this Paper is now closed.—Sec.

PUBLIC HEALTH DIVISION MEETING

8 May, 1956

Mr W. A. M. Allan, Member, in the Chair

The following Paper was presented for discussion and, on the motion of the ıairman, the thanks of the Division were accorded to the Author.

Public Health Paper No. 16

THE CONSTRUCTION OF MIDDLETON CONNECTING SEWER IN NORTH MANCHESTER

by

* Edwin Hibbert Collier, A.M.I.C.E.

SYNOPSIS

A portion of the sewer, which conveys sewage from the borough of Middleton to Man-ɔster Corporation's main-drainage system, was constructed in free air by direct labour tween 1946 and 1948, but work had to be abandoned owing to very difficult ground ıditions.

Construction under compressed air was resumed by contract in June 1950; the execution this contract forms the subject of the Paper.

Ground conditions were very variable, and compressed air at pressures up to 32 lb/sq. in. s employed extensively in both shafts and tunnels. In one tunnel excavation was ɔceeding in free air in a full face of very hard clay when water began to seep through roof a few inches in front of the leading edge of the shield. The miners were with-wn immediately and the tunnel was put under compressed air; when re-entry was de 20 min later about 70 cu. yd of sand was found to have entered the workings. The cing of a large cavity above the sewer and its subsequent grouting through thrust bores lescribed.

n another tunnel where the face was very large and open gravel, progress was reduced 3 ft in 24 hours because timbering had to be sealed with clay puddle to retain the ıpressed air.

Precast concrete segmental rings were used to line the shafts and tunnels; the shafts e at depths up to 105 ft below ground level with diameters ranging from 8 ft to 13 ft ., and tunnels were mainly 6 ft 5 in. diameter. The segments were lined subsequently h engineering brickwork to a finished diameter of 5 ft.

The contract, carried out on a prime-cost basis with incentives for speed and economy, completed in March 1952 at a cost of approximately £350,000.

Introduction

ɛ construction of the Middleton connecting sewer was authorized by the ıchester Corporation Act of 1920, to convey sewage from the borough of Middleton Manchester Corporation's main-drainage system. Owing to the abandonment of

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a proposal to construct a new arterial road along the line of which the sewer was to run, the borrowing powers sanctioned under this Act were not exercised.

Following the post-war decision of Manchester City Council to undertake large-scale housing development in the northern half of the city the sewer was re-routed to collect additionally the drainage from the new estates (Fig. 1, Plate 1). New borrowing powers were obtained and construction of the section of sewer between manhole 0 and manhole 6 was commenced in July 1946. This length of sewer was required to drain the first of the proposed housing estates and is, therefore, referred to as the first section, the remainder of the sewer between manhole 6 and manhole X constituting the second section. Construction of the first section was undertaken by the Direct Works Section of the City Surveyor's Department, which had extensive experience of sewerage works in the inter-war years.

Good progress was made between manhole 0 and manhole 2, which was executed by open cut in dry sand, but upstream from the latter, where the work was in tunnel, seams of water-bearing gravel and hard silt appeared in the face and working conditions became progressively more difficult. The tunnel driven upstream from manhole 2 eventually had to be abandoned, and further working shafts were established successively at manholes 3, 4, and 5, tunnels being driven in both directions from each of these shafts. With the exception of the tunnel driven upstream from manhole 5, which was entirely in hard silt, all drives had to be abandoned at distances ranging from 12 to 80 yd from the shafts, the final obstacle occurring when the hard silt was overlain by gravel near the top of the working face. In its natural state the silt was extremely hard and had to be removed with clay spades, but in contact with running water it became a slurry that flowed in through the timbering as soon as attempts were made to take the face forward. Since the cost of construction under these conditions was prohibitive, work was suspended in January 1948 leaving manhole 6 and 180 yd of the first section, in four short lengths, still to be completed, together with the whole of the second section, comprising 1,730 yd of 4-ft-3-in.-dia. sewer and six manholes at depths up to 105 ft below ground level.

Prior to closing down the work, 9-in.-thick brick headwalls were constructed at the end of each length of sewer, and the tops of manholes 3 and 5 were covered with heavy concrete caps to facilitate resumption of the work under compressed air. Since it was thought that manhole 4 might subsequently be used as a working shaft it was built with the shaft vertical over the centre of the sewer and the top was left unfinished a few feet below ground level to facilitate construction of a concrete air deck and vertical air-lock. The other manholes constructed by the Direct Works Section were of the side-entrance type, and were completed except for the provision of landings, ladders, and other ironwork.

Nature of contract for completion of sewer

Tenders for completing the works were invited from five contractors experienced in compressed-air tunnelling. Of these, three declined to tender and both the contractors who tendered based their quotations on executing the work in compressed air throughout, despite instructions that alternative rates should be given for any work found possible in free air. Since the prices appeared very high, it was decided to re-invite tenders on the basis of a target sum represented by the total of the priced bill of quantities, and the payment of a fixed fee adjusted by limited bonus or penalty depending on the actual cost and the period of completion. Kinnear Moodie & Co. Ltd were awarded the contract on this basis, the contract time being

FIG. 2.—RIB-AND-LAGGING CENTERING FOR CONSTRUCTING BRICKWORK LINING

FIG. 6.—VERTICAL AIR-LOCK AT WORKING SHAFT ON FIRST SECTION

FIG. 8.—JUNCTION OF 6-FT-DIA. TUNNEL WITH 4-FT-3-IN.-DIA. SEWER

FIG. 10.—FACE TIMBERING IN SANDY GRAVEL.

2 months for the completion of the first section and 27 months for the construction f the second section.

From the Resident Engineer's standpoint the execution of work on a prime-cost ontract enhances its interest, since in addition to his usual duties he must collaborate ith the contractor at all stages to ensure that economical methods are employed, pprove the purchase of all materials, agree wage differentials and bonus rates, check isposition and use of plant, authorize overtime working, and agree the allocation f all charges with the contractor's agent. These additional responsibilities, howver, make the Resident Engineer's task more exacting, and require additional erical and supervisory staff for checking and accountancy purposes.

Despite the care taken in preparing the contract documents numerous items of xpenditure were incurred which were not covered by the prime-cost and fixed-fee hedules. Where the amounts were small the charges were usually disposed of at te level, but to deal with other cases a suspense account was created in which ontentious costs were lodged and reviewed at periodical meetings. The total sum hich passed through the suspense account was surprisingly high, and illustrates hat it is impossible to cover every contingency in a contract document, however arefully worded.[1]

Tunnel lining

In submitting his tender the contractor suggested that the prime cost might be duced by using 6-ft-5-in.-internal-dia. tunnel linings for the second section and -ft-dia. linings to complete the first section, instead of 5-ft-3-in.-dia. linings pecified for both sections. The latter diameter was adopted by the corporation as e smallest permitting construction within it of a brick barrel to the required nished diameter of 4 ft 3 in., the annular space of 6 in. allowing for one ring of ngineering brickwork and a 1½-in.-thick collar joint.

The objection advanced by the contractor to the use of 5-ft-3-in.-dia. lining was hat it would allow room for only one miner at the face; after comparing costs with e 6-ft-5-in.- and 6-ft-0-in.-dia. linings it was agreed that the resultant savings from creased progress with the larger sections would more than offset the additional xcavation and material costs. There is no doubt that this decision to change to rger-diameter linings was right, particularly since progress, even with two miners in e face, was at times only 3 ft in 24 hours. Since the face worker in a compressedr tunnel may have behind him up to twenty men, with a battery of compressors splacing 2,000 cu. ft/min or more of air, the importance of even a small increase progress is evident. An additional advantage of the 6-ft-5-in.-dia. lining was that permitted the use of a tunnel shield, whilst it is large enough to accommodate two nes of 2-ft-gauge jubilee track.

Precast concrete segmental linings were used in preference to cast iron on the ounds of cost and availability.[2] From the ground-water levels found in trial orings it was expected that a maximum air pressure of about 20 lb/sq. in. would be lequate but, in fact, pressures up to 32 lb/sq. in. were found necessary. The uthor understands that the latter pressure is almost twice that to which precast oncrete linings had previously been subjected but there were no signs of failure while e tunnel was under air pressure, even though a number of segments had been acked owing to the erratic behaviour of the shield. Whether the segments would ave withstood an external pressure of 32 lb/sq. in. was not, however, demonstrated,

[1] The references are given on p. 719

since the length of tunnel where this pressure was used was lined with brickwork before the air pressure was taken off, to prevent possible loss of ground through the cracked segments. Pressures up to 24 lb/sq. in. were, however, used in other tunnels, and no defects were observed in the segments on return to atmospheric conditions. A difficulty experienced with the precast lining was the occasional "popping" of bougie plugs when the air pressure was taken off. It would be better therefore if segments used in compressed air could have a more positive plug than the tapered concrete ones used on this contract.

Grouting behind the tunnel linings was done with neat cement, the quantity of cement used per ring ranging from about 6 cwt in hard ground up to 30 cwt in open ground. Cement supplies were difficult to obtain throughout the job, and serious delays would have arisen had it not been possible to supplement the contractor's deliveries by drawing on the corporation's stocks. To alleviate the cement shortage a quantity of Blue Lias lime was obtained for grouting but the experiment was disastrous, the expansion of the lime on hydration causing extensive cracking of the segments, a number of which had to be taken out and replaced.

Segment joints were caulked with asbestos rope dipped in cement grout and hammered into the grooves, which were subsequently pointed with cement mortar incorporating a quick-setting additive. So long as the arrises of the grooves have not been broken in handling, the method provides an effective joint, but at damaged grooves extensive recaulking was sometimes necessary to effect a watertight seal. This problem would be reduced if the depth of the groove was increased from $\frac{3}{4}$ to, say, $1\frac{1}{4}$ in. since the extra depth would permit some damage to the arrises without the back of the groove being destroyed.

Brickwork barrel

As mentioned on p. 703 the original intention was to build a single ring of engineering brickwork to the required finished diameter of 4 ft 3 in. within 5-ft-3-in.-internal-dia. segments. Since the construction of a 4-ft-3-in.-dia. barrel within the 6-ft-5-in.-dia. segments actually used for the second section would have involved greatly increased quantities of brickwork, the diameter of the finished barrel on the second section was increased to 5 ft, the space between the segments and the inner ring of engineering brickwork being filled with common brick packing. On the first section since the existing lengths of sewer were of 4 ft 3 in. diameter it was desirable to adhere to this dimension in the new work; therefore 6-ft-dia. linings were preferred to 6-ft-5-in.-dia. linings to reduce the brick packing required.

As a result of these modifications the upstream section of the sewer is 9 in. larger in diameter than the downstream section, the change in diameter being made at a bellmouth with level invert situated between manholes 5 and 6 on the first section.

In constructing the brick lining the invert was laid a length ahead of the arch, the bricklayers' shift thus commencing with the construction of an arch followed by a length of invert (Fig. 2, facing p. 702). The advantages of working in this way were twofold, the construction of the arch in the first half of the shift allowing the laggings to be struck and the brickwork bagged-off before the finish of work, whilst the invert, having been built the previous day, was less liable to be damaged during the construction of the arch. Brickwork lengths were 8 ft 3 in. on the first section and 12 ft on the second section, the number of bricks per length being about 1,350 and 1,950 respectively. On the first section a length was laid by one bricklayer in a shift of 10 hours, and on the second section, where the larger diameter of the finished barrel allowed room for two bricklayers to work, a length was laid in about 8 hours.

In sewers of these relatively small diameters the use of radiused bricks is normally referred, but was not considered essential within the protection provided by the recast linings, and square bricks were used throughout. Difficulty was experienced n securing adequate quantities of engineering bricks and supplies had to be obtained rom five brickworks. Since this entailed considerable variation in size and quality, ricks from different works were used in separate sections of the sewer, the harder nd less absorptive bricks being reserved for the invert and the more absorbent ricks used in the arch.

Consideration was given to the use of in-situ concrete for the finished barrel, but n view of the difficulty of ensuring a dense concrete surface free from air-pockets nd honeycombing it was felt that brickwork would give more satisfactory results.

Manholes

The corporation had intended to provide six manholes on the second section, but ince the position indicated for manhole 1 was difficult of access, the contractor uggested that this manhole should be omitted and manhole 2 moved downstream o a position midway between manhole 3 and manhole 6 on the first section. This vas approved since the revised position of manhole 2 provided a very suitable site for he main working area (Fig. 3), although the distance of almost 400 yd to the adjacent nanholes is greater than would normally be desired.

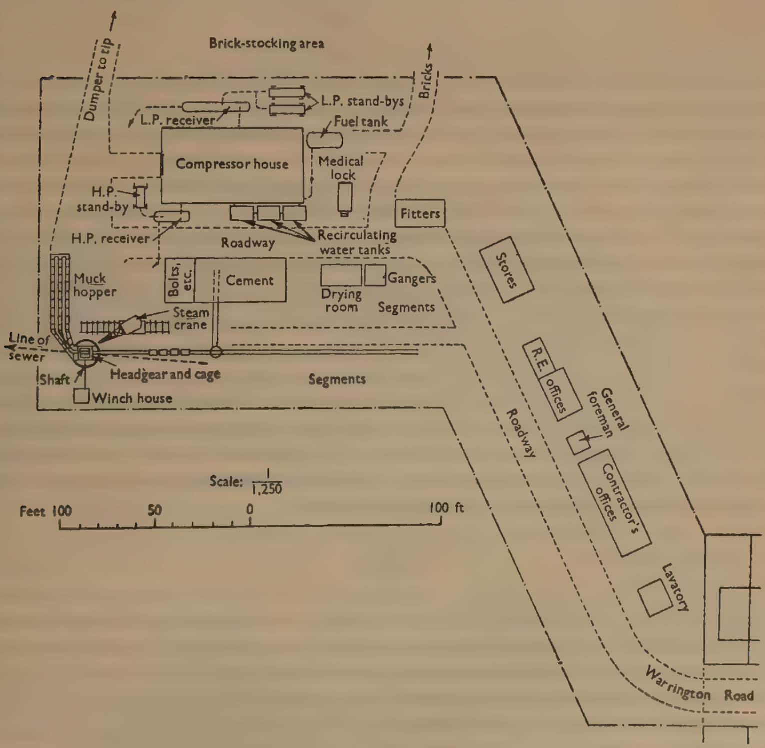

Fig. 3.—Layout at main working area on second section

In submitting his tender the contractor was asked to indicate his preference for either 10-ft-0-in.-internal-dia. precast concrete segmental shafts sunk directly over the centre-line of the sewer or 5-ft-6-in.-dia. brickwork shafts sunk off-centre with a side entrance. In all cases the contractor elected to use the former type, with the diameter increased to 11 ft to form a satisfactory connexion with the 6-ft-5-in.-dia. tunnel linings, the 10-ft-0-in.-dia. shaft having been specified for use with 5-ft-3-in.-dia. tunnel linings. Subsequently, it was decided to increase the size of the shafts at manholes 2 and 4 to 13 ft 6 in. internal diameter to enable a shield to be lowered down the former manhole and recovered at the latter. Accordingly, manhole 2 is of these dimensions, but as events transpired the shield had to be dismantled below ground and manhole 4 was constructed with a side entrance. Manhole 6 as eventually constructed was also of the side-entrance type, the diameters of the shafts of these two manholes being determined largely by the availability of segments at the time of construction. Typical details of central and side-entrance type manholes are shown in Fig. 4, Plate 1.

Details of manhole depths and shaft diameters are as follows:

Manhole No.	Internal diameter	Depth to invert
6: first section	8 ft 0 in.	51 ft 0 in.
2: second section	13 ft 6 in.	101 ft 0 in.
3: ,, ,,	11 ft 0 in.	105 ft 0 in.
4: ,, ,,	9 ft 0 in.	92 ft 0 in.
5: ,, ,,	11 ft 0 in.	58 ft 0 in.
X: ,, ,,	Special construction	10 ft 0 in.

The original intention was to line the precast shafts with one ring of engineering brickwork, but on comparing costs it was found that a 6-in.-thick lining of in-situ concrete would be cheaper, particularly since the contractor had available three sets of 10-ft-dia. steel shutters suitable for use in the 11-ft-dia. shafts. These shutters, which comprised four quadrant pieces of 4-ft.-9-in.-rad. with 6-in.-wide straight keys between were also used for the 13-ft-6-in.-dia. shaft, the 6-in.-wide key pieces being replaced by 2-ft-2-in.-wide keys to reduce the volume of concrete required. The thickness of the in-situ lining in manhole 2 thus ranges from 11 in. at the keys to about 6 in. opposite the mid-point of the quadrants, as shown in Fig. 5, Plate 2.

Landings occupying half the area of the shafts were provided at approximately 20-ft centres, chases being cut in the in-situ lining for their support. Ladders were of galvanized mild steel with $2\frac{1}{2}$-in. $\times \frac{5}{8}$-in. stringers and 1-in.-dia. rungs, fixed to substantial wall brackets with cadmium-plated bolts. Guard-rails were fixed along the edges of all landings and round ladder openings, the latter incorporating lift-bars which had to be raised before the ladder could be reached. Shaft cover slabs were located a few feet below ground level, access to the manholes being through a short brickwork shaft of 2-ft-3-in.-sq. internal dimensions.

Survey

First section

Records left by the Direct Works Section gave the chainages of the headwalls built at the ends of the isolated lengths of sewer, but unfortunately the steel bars left at ground level to mark the intersection points had either been removed or obliterated, and there was thus no surface record of the centre-line of the sewer.

Furthermore, when the lengths of sewer were entered it was found that centre-line marks scribed on iron dogs driven into the brickwork did not coincide with the centre

of the barrel by amounts up to 20 in. It became evident that the positions of the nds of the sewers would have to be established by underground surveys brought up o ground level through the manhole shafts. This involved setting up a theodolite n the sewer adjacent to a manhole, sighting on to a pair of wires suspended down he shaft, and turning the angles on to the headwalls, the location of the wires being imultaneously observed at ground level. The position of the headwalls thus obained had subsequently to be transferred to the new tunnel through a concrete air leck, and since the base line represented by the wires suspended down the manhole haft was only 2 ft 8 in. long there were obvious possibilities for error. In the ircumstances the achievement of four junctions between manholes 2 and 5 with a naximum error of $5\frac{3}{4}$ in. was considered satisfactory, the annular tolerance afforded y the 6-ft-dia. linings enabling a smooth junction of the completed barrels to be nade without any cutting of bricks.

Second section

On the second section independent closed-traverse surveys were made by the orporation and the contractor, using 6-in. micrometer theodolites reading direct to 10 sec with estimation to 1 sec. The results of the surveys showed very close agreement and the means of the two sets of values were adopted for tunnel driving. The esults achieved were entirely satisfactory, the main drives, one of which had negotiated two long curves, junctioning within less than 1 in.

In view of the defective condition of many of the properties adjacent to the line of sewer, photographs were taken before commencement of the work, accompanied by levels on thresholds, etc. The existence of the photographs, which were taken with the owners' knowledge and consent, probably acted as a deterrent to the making of claims, since these were limited to a small number, mainly in respect of alleged damage to interior decorations.

Completion of first section

Working shaft

It has been mentioned that manhole 4 was constructed with a view to possible subsequent use as a working shaft with a vertical-type air-lock. The contractor, however, did not wish to use a vertical lock, and included in his tender the construction of a new working shaft in a position midway between manholes 3 and 4, from which he proposed to tunnel on two faces through horizontal air-locks.

Excavating for this shaft, which was lined with 10-ft-internal-dia. precast concrete segmental rings, was commenced on 13 June, 1950, and sinking proceeded by underpinning without incident to a depth of 9 ft below ground level. Water was encountered at this level and some difficulty was experienced in building the next ring, since the ground, which was wet sand with thin seams of silt and gravel, tended to fall in before the segments could be got into position. After a further ring had been built under similar conditions the shaft settled about 8 in., at the same time moving 3 in. out of plumb. Because it now seemed unlikely that the shaft could be sunk by underpinning, attempts were made to continue sinking by building additional rings on the top of the shaft and loading with kentledge. By this means the shaft was sunk a further 2 ft 6 in. and the 3-in. tilt corrected, but it soon became evident that in the absence of a cutting edge, little further progress would be achieved by this method. Had the shaft been equipped with a cutting edge, it is probable that it could have been sunk in free air and the proposal to install horizontal locks and to

tunnel on two faces adhered to, but in the circumstances it was decided to suspend sinking while a concrete air deck was constructed and a vertical figure-8-type lock installed (Fig. 6, facing p. 702).

The air deck (Fig. 7, Plate 2) was designed to resist a pressure of 18 lb/sq. in. measured on the external area of the shaft, numerous 2-in.-dia. holes being provided through the 19-ft-square by 5-ft-thick deck to prevent any pressure build-up on the underside of the slab outside the periphery of the shaft. This pressure was a few pounds per square inch higher than the anticipated maximum requirement but, as an additional precaution, four of the five rings already built below the level of the air deck were lined with 8 in. thickness of reinforced concrete, the reinforcement extending into the air deck to tie the deck and shaft together.

Sinking was resumed by rotary shifts on 27 July, air being supplied by a mobile diesel-driven compressor with a displacement of 650 cu. ft/min, a similar machine being available as a stand-by. Difficulty was experienced in maintaining adequate pressure within the shaft, and consequently, excavation could often be carried out for only one segment at a time. At one stage the air loss was so serious that it was decided to block a number of the 2-in.-dia. breather holes, the additional depth of shaft constructed at this stage having increased the margin of safety against lifting of the air deck. Had additional low-pressure compressor units been available, these difficulties would probably not have arisen, but with the limited amount of plant then on site, the only means of holding back the water was to increase the pressure at the compressor to compensate for the drop at the working face. Consequently, pressures recorded at the compressor were frequently several pounds per square inch higher than the theoretical requirement for the head of water. The large quantity of 34 tons of cement was used in grouting behind the 20 ft of shaft constructed under compressed air; most of this was used in regrouting from time to time in attempts to reduce air losses.

Tunnel driving

The working shaft was bottomed on 18 August, excavation being taken a few feet below invert level to provide a 2-ft-6-in.-deep sump for the full area of the shaft. Compressed air was then taken off and the lower 10 ft of the shaft strutted. Before breaking-out into tunnel, a ring of 6-ft-dia. segments was built to correct line and level within the shaft on the upstream side; compressed air was then restored and the shaft lining broken out using the 6-ft-dia. ring as a templet. Tunnelling upstream commenced on 25 August and proceeded at an average rate of 6 ft in 24 hours until 5 October, when the tunnel was junctioned with the existing sewer downstream from manhole 4—58 yd from the working shaft. The junction is shown in Fig. 8, facing p. 703; the photograph was taken several weeks later, after the headwall left by the Direct Works Section at the end of the existing sewer had been broken out.

Ground conditions proved extremely variable (Fig. 9, Plate 2), the character of the face changing almost every foot, and it was necessary to timber the roof and face of the tunnel at all times. An indication of the method of timbering adopted is given in Fig. 10, facing p. 703. Steel piles 6 in. wide by $\frac{3}{8}$ in. thick by 4 ft long with the leading edges carried on an angle-iron rib were used to support the roof, the piles being levered forward with podger bars as excavation proceeded.

Tunnelling downstream from the working shaft commenced on 6 October, the headwall at the end of the sewer upstream from manhole 3 being reached on 1 November. The average rate of driving in this tunnel was about 7 ft in 24 hours.

Two 650-cu. ft/min mobile compressors were frequently required to maintain an

dequate supply of air in the tunnels, a third machine of similar capacity now being vailable as a stand-by. Air pressures varied from 10 to 14 lb/sq. in., the higher ressure being required when gravel occurred at the bottom of the face and the lower ressure in the last 20 yd of the downstream tunnel where the face was mainly hard ilt; only one compressor was required to maintain pressure in this length.

Concurrently with the sinking of the working shaft manholes 3, 4, and 5 were umped out to enable the existing sewers to be inspected and surveyed. Manholes 3 and 5 were pumped out without difficulty but the pumping-out of manhole 4 resented a serious obstacle, the water level, which was found 8 ft below ground urface, being lowered only 2 feet after a week's pumping with two 4-in. diaphragm umps. Eventually two powerful centrifugal pumps were employed for 4 days to ower the water level sufficiently to enable the sewer to be entered, the amount of umping required providing striking evidence of the difficulties under which the Direct Works Section had laboured in constructing this portion of the sewer.

When the sewer accessible from manhole 4 was entered, it was found that a umber of $1\frac{1}{2}$-in.-dia. tubes had been built into the brickwork at springing level, the ubes which had not been plugged providing a ready means of entry for ground-water. Plugging of the tubes considerably reduced the flow of water into the sewer ut caused numerous leaks to develop through the mortar joints. After unsuccessful ttempts had been made to seal these by caulking with lead wool it was decided to lefer further remedial work until the tunnels between manholes 3 and 4 had been lriven.

Other works carried out during the sinking of the working shaft were the construc-ion of concrete air decks at the bottoms of the shafts in manholes 3 and 4, and the hickening of the headwalls constructed by the Direct Works Section. As previously nentioned, the top of manhole 3 had been closed by a heavy concrete cap, but the ontractor felt that additional precautions should be taken to eliminate the risk of he shaft bursting when subjected to an internal pressure of the order of 15 lb/sq. in. The construction of a new air deck was not necessary at manhole 5, since the sewer ccessible from this manhole was not subjected to air pressure.

Following the completion of tunnelling between manholes 3 and 4, holes were lrilled through the headwall at the end of the existing sewer downstream from the atter manhole and the water in the sewer, which had again filled up, was run off to he sump in the working shaft. After draining off the water, the headwall was ompletely demolished, the section of sewer which had been constructed from man-iole 4 being then under air pressure. When the sewer was inspected a large volume of air was found to be escaping through voids in the mortar joints. Before de-nolishing the headwall upstream from manhole 4, the sewer was, therefore, grouted hrough holes drilled through the soffit at 10-ft centres. Approximately 10 cwt of rout per linear yard was used in this operation, which resulted in the leakage being ffectively sealed.

Tunnelling between manholes 4 and 5 was commenced on 17 November after the eadwall upstream from manhole 4 had been broken out. The rate of driving in his length was 9 ft in 24 hours, the increased rate of progress resulting from improved round conditions and on additional bonus incentive offered to the miners.

Tunnelling on the first section was completed on 12 January, 1951 when the tunnel lriven between manholes 2 and 3 was junctioned with the existing sewer upstream rom manhole 2. No difficulty was experienced with air loss in the existing sewer onstructed from manhole 3, but before breaking down the headwall downstream rom this manhole a test was made to confirm that the workings could withstand a

pressure of 17 lb/sq. in. and that this pressure could be maintained with one compressor. The maximum pressure actually required when tunnelling between manholes 2 and 3 was 15 lb/sq. in. except for a short period after the headwall was broken out, when a pressure of 16 lb/sq. in. was required with two compressors in operation.

In all tunnels segment joints above rail-track level were caulked as tunnelling proceeded to reduce air losses to the minimum. Invert joints could not be caulked until the rail track had been removed on completion of a section of tunnel, but on occasions when large air losses took place through these joints, sections of track were lifted during the week-end break and the leaking joints caulked before resumption of tunnelling.

Ancillary works

Brickwork lining was commenced early in February after the tunnels had been cleaned out and caulked; subsequently, the working shaft was filled in and the various items necessary to complete manholes 3, 4, and 5 were carried out. All work on the first section, with the exception of manhole 6, was completed on 26 May, 1951.

Manhole 6

In the light of events at the working shaft it was decided to commence sinking manhole 6 as a caisson. Initially ground conditions were favourable to this method of sinking, and the shaft was quickly taken to a depth of 18 ft through loam and wet sand. At this level the shaft moved slightly out of plumb, subsequent excavation disclosing that hard ground had been met on one side. After unsuccessful attempts had been made to continue caisson sinking, the shaft was grouted and an ordinary ring of segments substituted for the cutting edge. During this operation considerable ground was lost and the tilt of the shaft increased to 10 in. Sinking was continued by underpinning, but after two rings had been built in this way, work was interrupted by the breakdown of the crane. Since a major repair was found necessary, sinking was suspended pending the manufacture of a special tapered ring to level up the bottom of the shaft, which had now moved 12 in. out of plumb. The ring was available within a few weeks, but in view of other urgent commitments, it was decided not to continue work on manhole 6 at that time.

When work was resumed 7 months later, in February 1951, ground conditions had deteriorated to such an extent that it was impossible to make further progress by underpinning (Fig. 11, facing p. 718). After a number of alternative proposals had been considered, it was decided again to suspend operations until tunnel driving downstream from manhole 2 on the second section had been completed, when ground conditions at the shaft bottom would be known.

Following completion of this tunnel at the end of March, it was agreed that the logical procedure would be to complete the existing shaft under compressed air with a vertical air-lock. In view, however, of the high cost that this would entail, the corporation decided to sink a new shaft about 30 yd farther upstream where hard silt free from gravel and sand seams had been found in the tunnel, in the reasonable hope that this shaft could be got down in free air.

Accordingly, a new shaft was sunk 10 ft off the centre-line of the tunnel in 8-ft-dia. segments, the original shaft having been sunk centrally in 11-ft-dia. segments. The shaft was successfully sunk in free air during June 1951 (Fig. 12), but the occurrence of a 5-ft-thick seam of wet silt immediately above chamber-roof level made it advisable to use compressed air to construct the chamber. Air was conveniently supplied

rom the compressing plant at the main working area using the newly-constructed unnel downstream from manhole 2 as a pipeline, the horizontal air-lock used in riving the tunnel having been retained with this possibility in mind. As shown in ig. 7, Plate 2, the air deck in the manhole shaft was placed at a level suitable for a nding, and the necessity subsequently to break out the deck was thus avoided.

In view of these developments, manhole 6 was transferred to the second section for he purpose of completion-date assessment, and the contractor was consequently ntitled to 7 weeks' bonus on the completion of the first section, the contract time aving been extended by a few weeks to allow for extra works.

CONSTRUCTION OF SECOND SECTION

rogramme

It was proposed to carry out all tunnelling from working shafts at manholes 2 nd 5, with the converging drives meeting at manhole 4 where the shield to be used the tunnel upstream from manhole 2 would be recovered.

On this basis, the tunnel lengths were as given in Table 1.

TABLE 1

Working shaft	Direction	Termination of drive	Length: yards
Manhole 2 . .	Upstream	Manhole 4	710
	Downstream	Upstream end of existing sewer	400
Manhole 5 . .	Upstream	Manhole X	340
	Downstream	Manhole 4	280

By employing a shield (Fig. 13, facing p. 719) in the tunnel upstream from man-ole 2 the contractor hoped to achieve a rate of progress almost twice that in the ownstream tunnel. Thus, both tunnels would be completed at the same time and y necessity for single-face working avoided.

For reasons given later in the Paper this programme could not be fully imple-ented, and it became necessary to use manhole 3 as a working shaft for completing e shield drive. Driving with the shield was eventually continued for about 70 yd ostream from manhole 4, the length of tunnel driven downstream from manhole 5 ing correspondingly reduced.

As a result of this departure from programme, shield driving was continued for most 5 months after the tunnel downstream from manhole 2 had been completed, e extensive single-face working thus introduced adding appreciably to tunnelling sts.

ir-compressing plant

The main low-pressure installation was housed in a steel-framed building, ft × 40 ft in plan situated at manhole 2. Owing to the possibility of power cuts e use of electrically driven plant was not permissible at this working area, and the ree low-pressure units installed were driven by stationary water-cooled diesel gines. Two of the compressors had a displacement of 650 cu. ft/min and the ird machine displaced 1,000 cu. ft/min. All engines were provided with emergency l-storage tanks of 50-gal capacity fed from an elevated bulk-storage tank of

5,000-gal capacity. Cooling water for the engines was pumped into a header tank and recirculated.

At the secondary working area at manhole 5 immunity from power cuts was granted by the local electricity authority and an electrically driven stationary air-cooled compressor, displacing 1,123 cu. ft/min, was installed at this site. Stand-by plant common to both working areas was provided by a number of 650-cu. ft/min mobile diesel-driven compressors.

In general, low-pressure-air requirements at manhole 2 were met by two of the stationary compressors, but at one period the output of all three compressors, supplemented by two of the mobile machines, was required to hold up the face in the shield drive. At manhole 5, owing to the open nature of the ground in the downstream tunnel the stationary compressor had frequently to be assisted by a mobile machine, and occasionally a second mobile machine was required.

High-pressure air for bougie-pans, clay spades, and pumping, and for operating the hydraulic pump in the shield drive, was supplied mainly by stationary electrically driven compressors of 315-cu. ft/min capacity. These machines were supplemented as necessary by a number of 210-cu. ft/min mobile compressors, whilst in the event of a major breakdown the mobile low-pressure machines could be converted to high pressure, their displacement on high pressure being about 500 cu. ft/min.

Tunnelling from manhole 2

Tunnelling from manhole 2 was commenced on 16 August, 1950, after the working shaft had been sunk without difficulty in free air, the bottom 40 ft being in hard clay with occasional thin seams of sand or gravel.

In view of the favourable ground conditions found in the lower half of the shaft it was decided to defer the installation of air-locks so long as tunnelling could proceed in free air, the considerable depth of clay having aroused hopes that compressed air might not have to be used on the second section. Trial borings made before the contract was let indicated that the tunnels would be mainly in hard ground, but since the boreholes were few and put down at distances up to 100 ft from the line of sewer their value was very limited in ground of the variable nature known to exist in north Manchester.

Tunnelling was started on the upstream face, the downstream drive being commenced a few days later when sufficient length was available in the upstream tunnel to allow a rail cross-over to be installed and a raft of muck skips accommodated clear of the shaft bottom. Ground conditions in the upstream tunnel continued favourably for 60 yd from the shaft when thin seams of water-bearing sand appeared in the face. The rather difficult working conditions thereby created brought to a head the recent dissatisfaction of the miners at what they considered to be inadequate bonus rates, and the face workers from both this tunnel and the downstream tunnel, which had run into similar difficulties only 17 yd from the shaft, left the works without notice. In these circumstances, after boreholes had revealed continued wet ground ahead, it was decided to box up the faces and install air-locks in both tunnels (Fig. 14).

Tunnelling in the downstream tunnel was resumed in compressed air on 11 October, with a gang of miners recruited from the London area. After a satisfactory basis for bonus had been established steady progress was maintained in this tunnel, the rate of driving averaging 9 to 12 ft in 24 hours. Apart from a length of a few yards where the face was in hard clay the tunnel was constructed under compressed air throughout, with pressures ranging from 16 to 24 lb/sq. in. An unusual feature was the time taken to make the segments watertight, re-caulking and re-grummeting

aving to be continued for 10 weeks after driving had been completed. Since this roblem did not arise to the same extent in other tunnels, the trouble was probably ue to failure of the caulker to carry out his work properly, the extent of the defective ork not becoming evident until the air pressure was reduced after the invert had een cleaned out.

Driving in the upstream tunnel was resumed on 22 October under an initial ressure of 11 lb/sq. in. Later in the day the pressure had to be raised to 20 lb/sq. in. dry the face, but after 10 yd had been driven in compressed air the face was again ntirely in hard clay and the air pressure was taken off. After tunnelling had roceeded for a further 8 yd wet sand appeared in the roof and compressed air had be restored at a pressure of 20 lb/sq. in. During the next 2 days the pressure was rogressively increased to 26 lb/sq. in. as the nature of the sand changed and the epth of sand in the face increased. This pressure was not sufficient, however, to old back the water, and a position was soon reached when the shield was full of nd, which entered the tunnel faster than it could be removed. Since the air-ck had been tested to a pressure of only 25 lb/sq. in. it was considered inadvisable increase the pressure in the tunnel beyond 26 lb/sq. in., and the face was, therefore, aled off by removing the sand from the shield and filling the space between the aphragms with straw and puddle clay. This work was carried out under great fficulty, and was one of those operations which, by some means or other, men are ble to perform in the face of dire necessity, but which defies subsequent description to how it was done.

The difficulty of this situation was increased by the recent erratic behaviour of e shield which had commenced to climb when tunnelling had progressed about yd from the shaft; the climb continued to such an extent that at 50 yd from the aft the invert of the segments was above the designed level of the back of the inner ng of engineering brickwork. In attempts to get back to gradient the shield was oved on the top three rams only, and stout pieces of timber were wedged between e bottom of the forward diaphragm and the hard clay face to produce a downward t. These manœuvres were partially successful but unfortunately fractured a umber of segments owing to the excessive ram pressures exerted. When the sand as reached even greater pressures were employed to shove the shield forward against e resistance of boards which had been placed in front of the forward diaphragm to old back the sand. In addition to difficulties at the face itself there was, therefore, e added problem of fractured segments through which water and sand were tering the tunnel, or, alternatively, compressed air was escaping, depending on e nature of the ground behind the damaged segments.

After sealing the face, the air-lock was reinforced and tested to a pressure of lb/sq. in. Tunnelling was then resumed under a pressure of 30 lb/sq. in. which as initially sufficient to dry the face. Within a few hours, however, sand again tered the tunnel through the uncaulked invert joints, and the lower half of the face gan to run. After six rings, i.e., 12 ft, had been built under these conditions, the ce was again closed and the pressure raised to 32 lb/sq. in. to enable the invert to cleaned out and caulked. At the same time the 95 yd of tunnel constructed was grouted and attempts were made to caulk the fractured segments. One ring that d been particularly badly damaged was taken out and replaced.

These operations improved conditions considerably, and shortly after tunnelling as resumed on 22 November, progress reached 8 ft per shift, excavation consisting erely of shovelling up the sand as the shield was pushed forward. Unfortunately, is rate of progress was not achieved on each shift because, owing to difficulties with

the rams, very high pressures, of about 2,000 lb/sq. in. or more, had frequently to be used to shove the shield, with consequent damage to segments and loss of time in replacing any that were badly fractured. Notwithstanding these delays, an average rate of about 5 ft per shift, i.e., 15 ft in 24 hours, was maintained in tunnelling through the sand, which continued for a further 65 yd when the face again came into hard clay. Air pressures used to get through the sand ranged between 32 and 25 lb/sq. in., the former being required for a distance of about 35 yd.

The remainder of the drive upstream from manhole 2 was comparatively uneventful, the ground being mainly hard clay which would normally have been excavated in free air. When the air pressure was reduced below 16 lb/sq. in., however, water and sand entered the tunnel through the cracked segments, and when the pressure was lowered to 12 lb/sq. in. a number of bougie plugs "popped" and sand gushed in under considerable pressure.

In the light of these circumstances, which necessitated keeping the tunnel under compressed air, irrespective of conditions at the face, it was decided to discontinue driving from manhole 2 at a point 20 yd upstream from manhole 3, and to use the latter manhole as a working shaft for the completion of the shield drive.

Accordingly, driving upstream from manhole 2 was terminated on 16 March, 1951, and a 4-ft-thick brick headwall was built in the tunnel 15 yd downstream from manhole 3. The tunnel downstream from this headwall was kept under air pressure until the length driven through running sand had been lined with brickwork. This operation, including subsequent grouting of the brickwork through tubes built into the soffit at 10-ft centres, was completed on 18 May. The remainder of the tunnel upstream from manhole 2 was brick-lined in free air concurrently with the downstream tunnel, a section of which was made ready for brickwork on 26 May, after extensive recaulking as referred to on p. 713.

Tunnelling from manhole 3

Excavating for manhole 3 commenced on 2 March, 1951, the shaft being junctioned with the tunnel, which was now in free air upstream from the headwall, on 2 April. This shaft was exceptional in that the ground was dry throughout, the lower 40 ft, as at manhole 2, being again in hard clay.

Tunnelling commenced in free air on 9 April and continued without incident in hard clay until the end of the month when, in view of the conditions being encountered in the drive downstream from manhole 5, it was considered advisable to construct an air-lock in the tunnel a short distance upstream from manhole 3. After a short delay while this work was carried out, tunnelling was resumed in free air, progress being at the very satisfactory rate of about 18 ft in 24 hours.

At 3.50 p.m. on 12 June, when the face was within 38 yd of the position of manhole 4, water broke through the roof of the 2-ft length of tunnel being excavated ahead of the shield. In accordance with instructions prepared in anticipation of deteriorating ground conditions, the miners withdrew from the tunnel and compressed air, brought overland in two 6-in.-dia. pipes from the compressor house at manhole 2, was turned on, the door holding fast at 4.0 p.m. On re-entry at 4.20 p.m., when the air pressure had reached 10 lb/sq. in., it was found that an estimated volume of 65–70 cu. yd of sand had entered the tunnel, completely burying the shield, shield pump, and bougie pan, and extending approximately 40 yd back from the face. Access to the face was gained next day, when it was discovered that the sand had entered the tunnel through a hole 4 in. in diameter. Exploration of the hole disclosed that the

ay roof was only 18 in. thick at this point, and was overlain by sand which, under r pressure, appeared quite dry and firm.

Since the tunnel at this point was under the centre of an important highway, the ad was closed on 14 June after the necessary traffic diversions had been arranged. unnelling was resumed the same day under an air pressure of 15 lb/sq. in., and was ntinued for a further 20 yd to provide working space for exploratory operations. he face was then boxed up and thrust bores in the form of 2-in.-dia. steel tubes were cked upwards through ring 1012, where the ground had been lost (see Fig. 9, late 2).

routing of cavity under Victoria Avenue

The existence of a cavity above ring 1012 was discovered on 24 June, the pressure quired to jack up the tubes clearly indicating that the cavity extended from 7 to ft above the top of the tunnel. Information provided by further bores through ngs 1010 to 1014 was conflicting, and it is probable that falls of ground were taking ace since the roof of the cavity, so far as this could be judged from within the nnel, seemed to be getting progressively higher, whilst the cover over the tunnel as increasing in depth.

The thrust bores were withdrawn on 27 June and new tubes, provided with a umber of slotted holes in the top 4-ft lengths, were jacked up at rings 1011 to 1014. total of 32½ cu. yd of grout was injected into the cavity through these tubes before e highest tube, which had been pushed to a height of 39 ft, refused. Since this as considerably less than the volume of ground lost, further tubes were jacked up rings 1009 and 1015. The latter tube would not go higher than 35 ft above the nnel, where presumably it came into contact with the hardened grout, but the tube ring 1009 was raised to 48 ft and took 19½ cu. yd of grout, making a total for all bes of 52 cu. yd.

Bearing in mind that some bulking of the original volume of sand may have curred both in the tunnel and when the falls took place, the cavity was now garded as satisfactorily grouted and tunnelling was resumed. After allowing a rther few days for possible subsidence to develop, the road was opened to traffic on July, 1951. No settlement has since taken place.

unnelling from manhole 5

This shaft was sunk in free air between 22 October and 18 November, 1950, tunnel iving being commenced on both faces on 22 November. Compressed air at essures between 3 and 10 lb/sq. in. was used to drive the last 70 yd of the upstream nnel, the first 270 yd being driven in free air through hard clay at a rate hardly rying from, and never exceeding, 12 ft in 24 hours. The comparative rates of ogress achieved in this tunnel and in the tunnel upstream from manhole 3, where ound conditions and incentives to progress were identical, is interesting, progress in e latter tunnel, as previously stated, averaging 18 ft in 24 hours, with the maximum, hieved on one occasion only, of 24 ft. The shield was not a factor in this comrison, since it provided no advantage in the hard clay, the facility of easier building the tail being offset by the time taken to shove.

The tunnel downstream from manhole 5 had been driven in free air for 79 yd from e working shaft by 17 January, 1951, when water commenced to percolate into the nnel a few feet back from the face. The flow of water quickly increased, and after hole bored through the face had indicated the apparent presence of running sand ead, it was decided to close the face and discontinue tunnelling until compressed

air was available. While the face was being boxed up the flow of water again increased and a considerable quantity of sand was washed into the tunnel. The flow of water continued until 8.0 p.m. on 18 January and then suddenly stopped, by which time about 20 cu. yd of ground had been lost. During this time the construction of a cofferdam, 8 ft back from the face, had been put in hand; this was completed on 19 January, and the space between the dam and the face was filled with puddle clay to safeguard against the loss of further ground.

Tunnelling was resumed on 2 February, under an air pressure of 15 lb/sq. in. After the first few yards where the ground was composed of layers of sand and clay, a full face of open gravel was encountered, the marked absence of fine material giving rise to conjecture that the sand washed into the tunnel might formerly have occupied the voids, a theory supported by the absence of any subsequent subsidence. The nature of this ground, which persisted in varying degrees throughout the remainder of the drive (Fig. 9, Plate 2) necessitated the timbering of the entire face, roof, and sides, and the "pugging" of all spaces between boards to retain the air within the tunnel, water flowing in very quickly when the face was taken forward. The resulting slow rate of progress, which was initially 3 ft in 24 hours, gradually increasing to 6 ft as experience was gained, made the work very costly, and at the end of May, when tunnelling had progressed only 210 yd from the working shaft, it was decided to close down the face and to complete the remaining 70 yd to manhole 4 with the shield. This decision involved the abandonment of the proposal to recover the shield at manhole 4, but made possible the reduction in diameter of this manhole from 13 ft 6 in. to 9 ft.

Manhole 4

As Fig. 9, Plate 2, indicates, hard clay was found in the tunnel at the position of manhole 4, but the occurrence of water-bearing sand at tunnel-roof level within a short distance on either side gave cause for concern that difficulty might be experienced in getting the shaft down.

Excavation was commenced on 9 August, 1951, and the shaft was quickly sunk through dry sand to a depth of 63 ft when the occurrence of water prevented further progress by underpinning. Since the corporation was anxious to avoid the use of compressed air at this manhole, the contractor suggested an ingenious method of continuing sinking in free air using a modified form of caisson. In principle, the scheme was to break out sufficient depth of lining at the bottom of the shaft to permit the building of a short caisson, of the same diameter as the shaft, below the underside of the lowest ring left in. The caisson would be driven down to firm ground by hydraulic jacks bearing against the underside of the lining, additional rings being added in the usual way as sinking proceeded.

After certain precautionary measures had been taken, including the trussing of the shaft from ground level by wire ropes fastened to the segments, the caisson was installed as shown in Fig. 15 and jacking commenced on 6 September. Conditions initially were favourable, and the caisson had been driven down 7 ft without appreciable loss of ground when an obstacle was met on one side. Probing of the bottom indicated that hard ground had been reached, and that the hard surface sloped about 2 ft 6 in. across the width of the shaft. After efforts made to drive the caisson into the clay had proved unavailing, attempts were made to excavate down to the cutting edge with a view to digging out the clay. This operation was proceeding, to the accompaniment of considerable loss of ground, when the caisson settled and a circumferential crack developed in the shaft lining 14 ft above the bottom ring.

The possibility that the surface of the clay might dip had been considered when ıe scheme was formulated, but preparations against this had unfortunately not een made in sufficient time to deal with the situation when it developed. In the

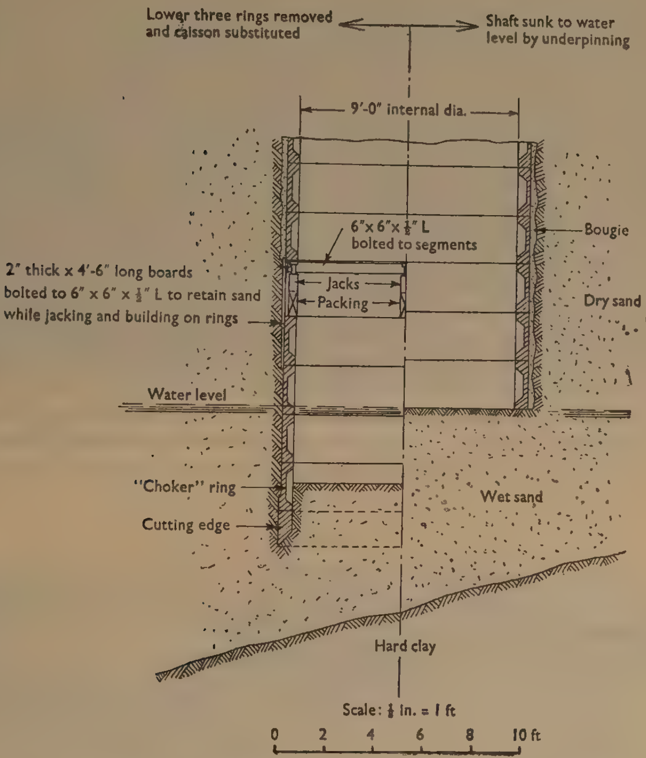

FIG. 15.—HALF-SECTION THROUGH MANHOLE 4 SHAFT SHOWING CAISSON SINKING

·cumstances, there seemed no alternative but to backfill the caisson to prevent rther loss of ground, and to complete sinking in compressed air, using a vertical ·-lock.[3]

COMPLETION OF CONTRACT

The contract was completed on 22 March, 1952, the last item being the successful outing of the 4-ft-3-in.-dia. sewer upstream from manhole 5 on the first section, ıere considerable percolation of water was taking place through defective brick-rk joints.

Notwithstanding the adverse conditions, the works were thus completed several onths ahead of contract time, and the drainage requirements of the new housing ates duly met.

Labour conditions

The frequent variations in ground conditions and the alternation of work between free and compressed air, with consequent changes in the length of shift, created a problem in the fixing of bonus rates which persisted throughout the work. In the early stages the corporation was understandably insistent that the incentive rates should be related to the allowances made for excavation in the priced bill, but this ideal had to be abandoned when it became apparent, as evinced by the walk-out of the miners from the tunnels at manhole 2, that upward revisions would have to be made in order to achieve progress and retain suitable labour on the works.

Expenditure in excess of the estimated prime cost was thus incurred shortly after the commencement of work, the rate of over-spending increasing sharply when it became necessary to employ air pressures of about 30 lb/sq. in. During this period consideration was given to reducing the length of shift to 6 hours, but in view of the prevailing labour shortage and the probability that the adverse conditions would not persist, the 8-hour shift was retained.

Numerous cases of compressed-air illness occurred, particularly during the period when high pressures were used in the tunnel upstream from manhole 2. So far as the Author is aware, no permanent injury was suffered by the personnel concerned, who were treated in a medical lock brought to the site in the early stages of the job.

Labour turnover was considerable, more than 1,000 men passing through the books during less than 2 years. Since a maximum of 190 men was employed at any one time the turnover, which affected all classes of workers, was about 300%.

Costs

The cost of the works was £344,000, representing an expenditure of approximately 25% more than estimated, after allowing for extra works and increased costs owing to rises in wages and materials from the rates ruling when the contract was let.

The allocation of prime-cost expenditure was as follows:

		%
Labour		45·0
Materials:	%	
Segments and accessories	18·8	
Bricks	3·3	
Cement	3·3	
Timber	1·7	
Sand and aggregates	0·8	
Miscellaneous	2·1	
		30·0
Plant hire, including haulage		18·0
Fuel and power		5·5
Installation of services		0·5
Unclassified		1·0
		100·0

Acknowledgements

The Paper is presented by permission of Mr Rowland Nicholas, C.B.E., B.Sc. (Eng.), M.I.C.E., City Surveyor and Engineer of Manchester, under whose direction the work was carried out.

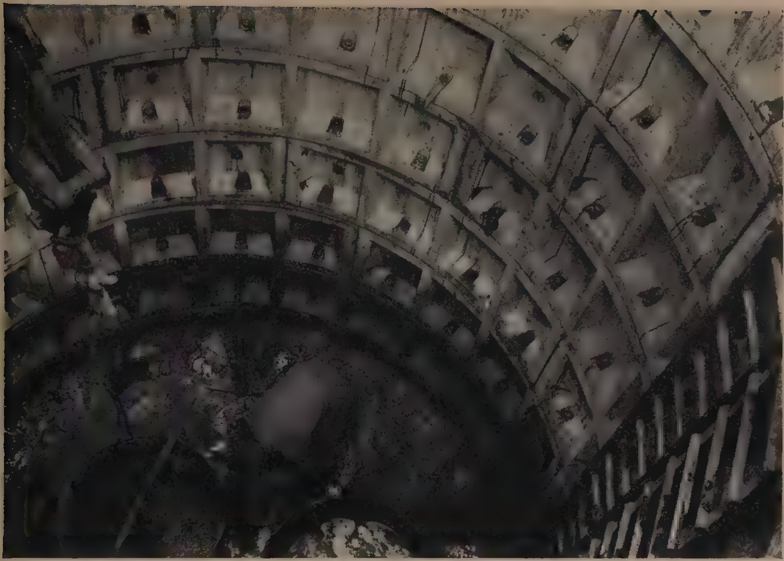

FIG. 11.—EXCAVATING IN WET SILT AT MANHOLE 6

FIG. 12.—SINKING MANHOLE 6 IN NEW POSITION

Fig. 13.—7-ft-6-in.-dia. tunnel shield

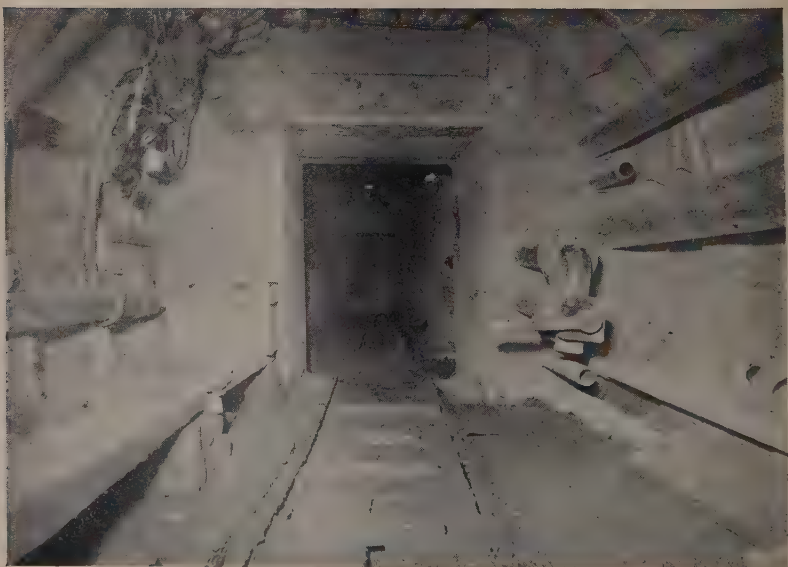

Fig. 14.—Horizontal air-lock upstream from manhole 2

THE CONSTRUCTION OF MIDDLETON CONNECTING SEWER IN NORTH MANCHESTER

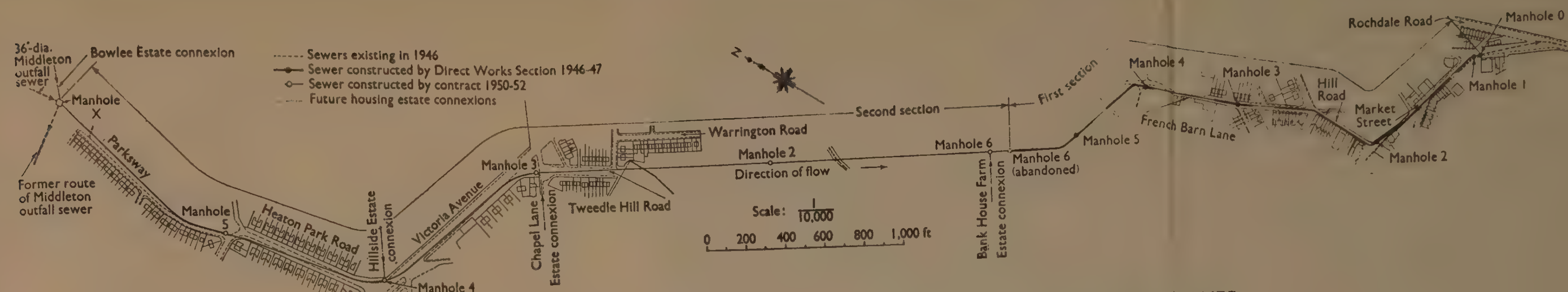

FIG. 1.—ROUTE OF MIDDLETON CONNECTING SEWER

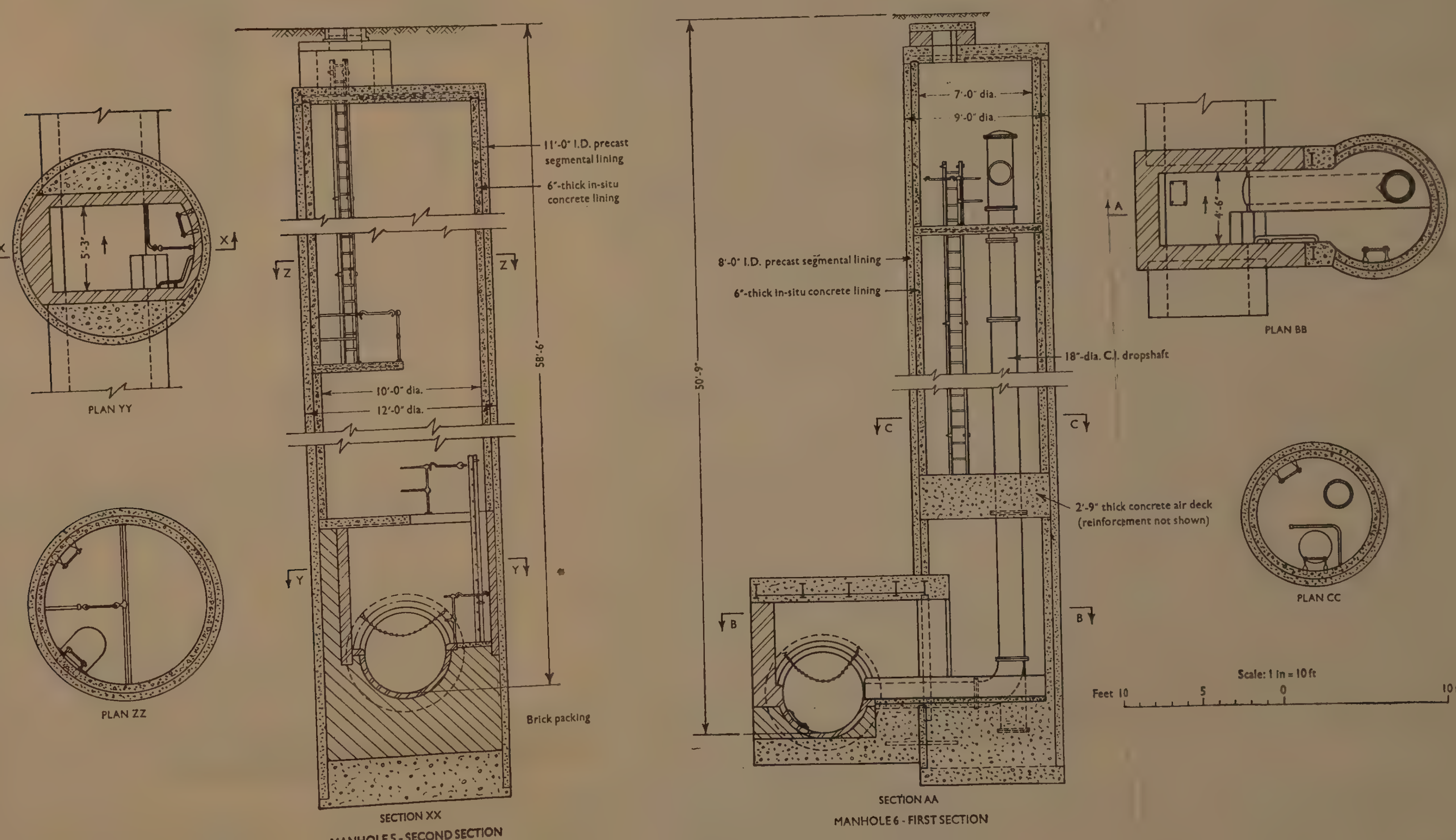

FIG. 4.—TYPICAL MANHOLE DETAILS

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THE CONSTRUCTION OF MIDDLETON CONNECTING SEWER IN NORTH MANCHESTER

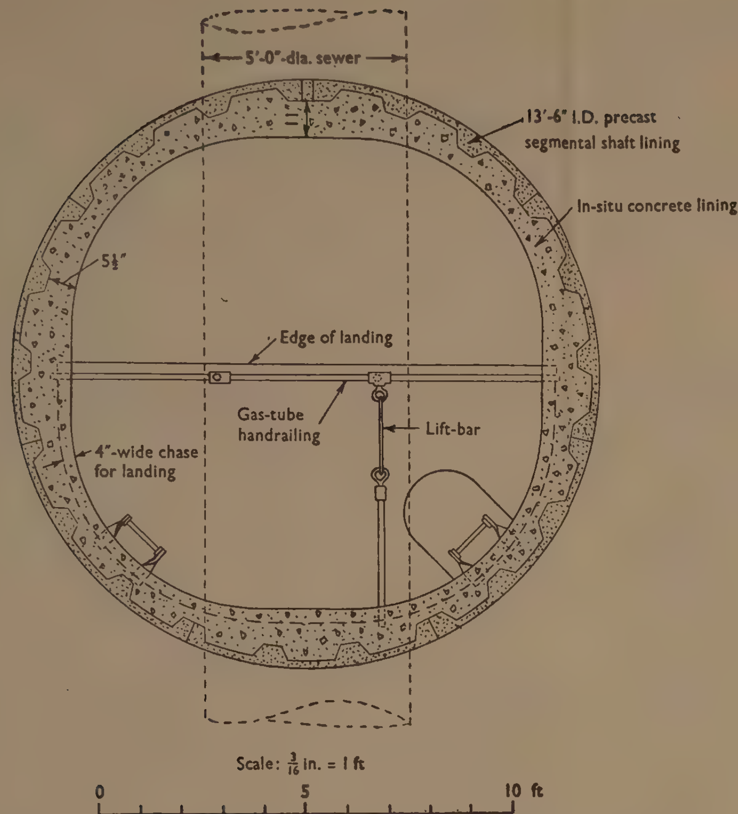


Fig. 5.—Section through Manhole 2 Shaft Showing Lining and Typical Landing Detail

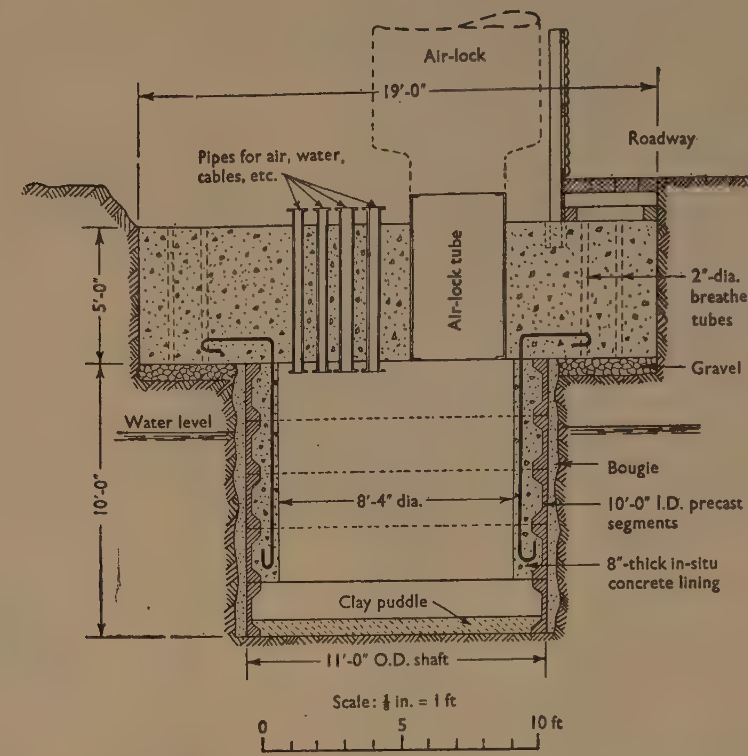
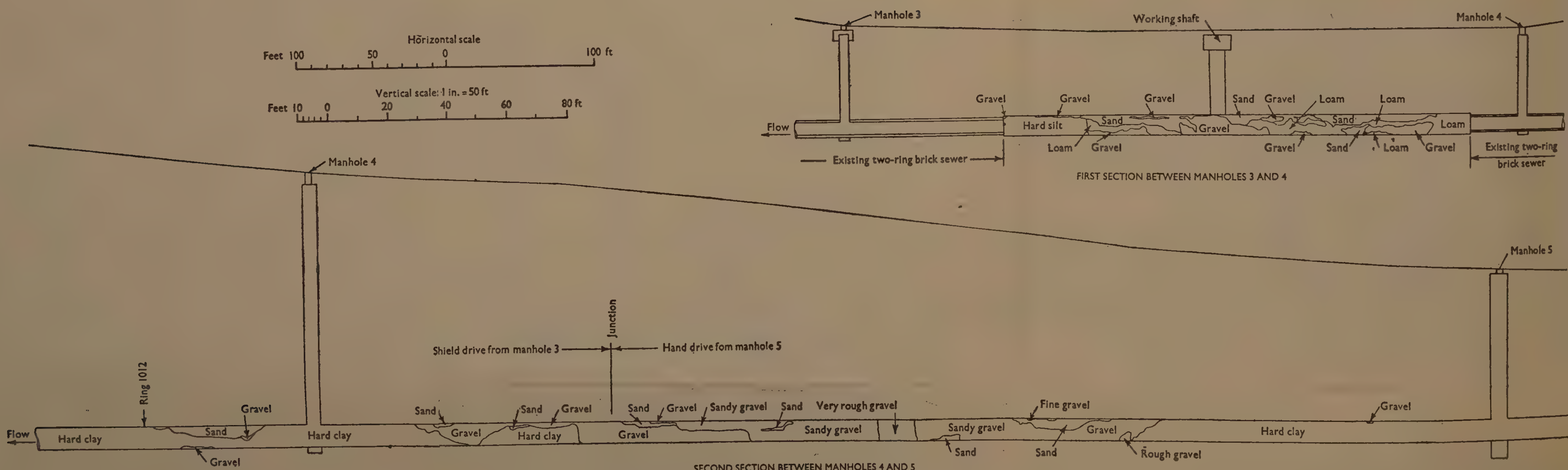


Fig. 7.—Air Deck at Working Shaft on First Section



E. H. COLLIER

Fig. 9.—Longitudinal Sections Showing Ground Strata

William Clowes & Sons, Limited: London

Kinnear, Moodie & Co. Ltd were the contractors, Mr Peter Murray, B.Sc., M.I.C.E., eing Director in charge of the work, with Mr J. S. Cooke acting as Site Agent.

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The Paper, which was received on 21 February, 1956, is accompanied by ten hotographs and seven sheets of drawings, from some of which the half-tone page lates, folding Plates 1 and 2, and the Figures in the text have been prepared.

Discussion

The Chairman, in proposing a vote of thanks to the Author, said he thought it was a ery good thing for the Institution to have before it for discussion a Paper which described e carrying-out of a difficult job in difficult ground and which went into some detail in escribing how those difficulties were overcome.

The Paper stated very clearly something which had been laid down by the late Sir aurice Fitzmaurice when he was Chief Engineer of the London County Council. He d said that one should never construct a tunnel that was too small to employ two miners the face, which was a very good axiom so far as tunnel driving was concerned. The uthor had proved that in Manchester a tunnel small enough to employ only one miner the face required practically the same organization from face to surface as a larger nnel giving room for two miners to work; therefore by adopting the larger size extra pacity was obtained at little extra cost.

The Chairman thought Manchester Corporation had been right in deciding to line the nnel with brickwork, particularly the invert. His experience had been that sewers ould be built in brickwork, but if that was not possible a brick-lined invert should be ovided. Those who had had experience of constructing sewers where the gradient had necessity to be slack would know the difficulty in obtaining a true invert gradient if ncrete were used, whereas with brickwork, if skilled bricklayers were employed and od engineering bricks used, a very nearly perfect gradient could be obtained no matter w flat the sewer; a well-graded brick invert lessened the possibility of silting and reduced intenance.

When the work had been started by the direct works organization in Manchester, they d apparently run into considerable difficulties. It would appear that the work had ginally been undertaken with inadequate ground survey. There was a statement in Paper that no sooner had they started to drive a tunnel than they had had to abandon because they ran into bad ground. Why were they not aware that that bad ground sted? It was normal, before carrying out works of that nature, to make a very com-hensive ground survey by means of boreholes, or even trial shafts. Although a bore-e did not always indicate the true nature of the ground in its vicinity, it gave some ication of the sort of troubles that might be expected, particularly if the use of npressed air was contemplated. The importance of obtaining the most accurate

information regarding ground conditions could not be overemphasized. Where the boreholes indicated there was likely to be trouble from bad ground the Chairman favoured checking the conditions by means of trial holes.

In connexion with the survey, when the work had been resumed and the original workings examined, the centre-lines that had been put in the tunnel for the purpose of carrying on the drive were apparently found to be up to 20 in. off centre. That seemed to be an extraordinary state of affairs; perhaps the Author could give some explanation.

The Paper went into some detail about the survey work. He thought that the engineers were to be congratulated on the fact that in those difficult circumstances they had finished up with an error within 1 in.

Those who had had to do any tunnelling work were familiar with the creep of water along the outside of a tunnel and the ways and means of stopping it. Very often tunnels constructed in perfectly dry blue clay eventually became wet because the working shaft had to be driven through a water-bearing strata and adequate measures to prevent the water seeping down the shaft and along the outside of the tunnel had not been taken.

With regard to the method by which the work had been carried out, the Paper stated that when it was decided to restart the work five firms were invited to tender and only two of them apparently had submitted a fixed-price tender. Since the Manchester Corporation considered the prices submitted were too high, they had decided to seek fresh tenders on the basis of a target-type contract and the work was eventually carried out in that way. That type of contract put a great deal of responsibility on the Resident Engineer who had got to administrate as well as supervise the job.

The Paper stated that the final cost of the job was £344,000. It would be interesting to know whether the final cost of the job worked out more than the original fixed-price tender and whether that method of carrying out the work had proved satisfactory.

Mr Peter Murray (Joint Managing Director, Kinnear Moodie & Co. Ltd) remarked that to a civil engineer Manchester's chief source of notoriety was the complete unreliability of its subsoil. The contractors in the present case had, about 20 years previously, built the main sewer along Rochdale Road, and they were, therefore, well aware of the difficulties a tunnelling contractor could expect.

The Author had described how the ground changed from week to week and sometimes almost from yard to yard, which made for delays in changing from one technique of tunnelling to another technique. The completion of the first section had been very slow and intricate, and the survey of the old work had not been the least of the difficulties. Whatever one might think of the original attempt at constructing the first section, he thought one must be full of admiration for the way in which those concerned had battled on in building a brick barrel in what must have been cascades of water.

The Author had hinted that the shield was hardly justified, and in that he must agree. The dry silt that they had run into had been too hard to be able to shovel the cutting edge through without breaking the concrete segments behind, and that had made it necessary for excavation to proceed ahead of the cutting edge. It was that, perhaps, which had contributed to the severity of the run-in that had occurred in the second section under Victoria Road, because the protection from the hood of the shield had been of no use.

To the contractors, the treatment of that subsidence had been perhaps the most interesting feature of the job. It had been known that 60–80 cu. yd of mud had run into the tunnel, and the location of that "run-in" had been exactly underneath a very big road artery. After closing the road, the first idea had been to sink a trial hole and find out something about the cavity, but then it had been suggested that, since compressed air had been put into the tunnel since the "run-in", the cavity would be full of compressed air and might conceivably be holding up the roadway. Therefore, permission had been sought to try to fill up the cavity from the tunnel, in order to avoid bringing in the roadway, which a trial hole might have done. It had been a pretty long chance, but, as was stated in the Paper, they had got 52 cu. yd of grout into the cavity, which was 48 ft above the tunnel, and he believed that it had taken about 3 weeks.

The high air pressures (up to 30 lb/sq. in.) had been quite unexpected. That could not

related to any water level shown on any of the boreholes and it was worth considering hether, in repelling mud and very finely grained material, one was not dealing with a iid weighing perhaps 90 lb/cu. ft instead of water which weighed 62 lb/cu ft. It was, course, another confirmation of the well-known axiom that any evidence given from anchester boreholes should be looked on with the gravest suspicion!

The Author's comments on reinforced concrete segments were much appreciated. here was always a danger of cracking concrete segments when a shield was used, pecially if the shield got off line, in which case more pressure had to be put on one part the ring than another. Mr Murray could not see any way of avoiding that without creasing the costs of the segments enormously.

The blowing-out of grout holes owing to the external pressure had caused difficulty, and was the first time that the contractors had encountered that. There again, the safe lution would be to have screwed sockets cast into the skin of the segment, to grout, and ien to screw in the grout plugs, but that was quite an expensive proposition and might it £1 or £2 on the cost of a ring. Alternatively, where high pressures were expected, the ze of the grout hole could be reduced and a specially small gun used to inject the cement. ut where a tunnel was to be lined internally afterwards, it would probably be quite tisfactory to hammer in soft wooden plugs instead of concrete plugs.

The Author mentioned that there had been numerous cases of compressed-air illness. e believed that there had been twenty to thirty in all, but fortunately only one had been ostinate and none serious.

With regard to the letting of the contract on a prime-cost basis, when it was considered at two-thirds or three-quarters of the tunnel had been driven in compressed air, it as questionable whether there had been any saving at all and whether the Manchester orporation would not have been well advised to accept a fixed price. The carrying-out that type of work on a prime-cost basis was unsatisfactory from a contractor's point of ew, because of the delays which occurred. For instance, they had run into a gravel face stead of a clay face. The miners had immediately asked for a new rate; they could do ly one ring instead of two, so they wanted twice as much money. Instead of the ntractor being able to settle that question it was necessary to go to the Resident ngineer, and he would have to go to his higher authority in the Town Hall.

As the Author had mentioned, the contractors had saved the Corporation a great deal worry by offering a lump sum price for the hire of plant. It would have been almost npossible to have computed what plant was idle and what plant was working, and how any hours it had worked, on a job of that description.

Mr D. H. Hughes (Senior Engineer, J. D. and D. M. Watson) said that during the ıst 4 or 5 years he had been associated with the construction of the Lee Valley Low Level wer, which was one of the trunk sewers on the East Middlesex Main Drainage Scheme. bout 3 miles of that sewer, of 5 ft 6 in. and 4 ft 6 in. internal dia., had been built in nnel through varied ground not unlike that met at Manchester, although the changes strata had not been so abrupt. He had noted one or two points of comparison that ight be of interest.

The Author was emphatic that building a 5-ft sewer in 6-ft-5-in. or 6-ft rings was a tter proposition than building a 4-ft-3-in. sewer in 5-ft-3-in. rings. He would say that at was contrary to his own experience. In the case Mr Hughes was describing the ft-6-in. sewer in 6-ft-5-in. rings, which might be expected to be less costly than a 5-ft wer in rings of a similar size, had cost about 10% more than the 4-ft-6-in. sewer in 5-ft in. rings.

In the best working conditions, which had been in compressed air on the smaller tunnel, e maximum advance had been almost the same as the 18 ft in 24 hours mentioned in e Paper. Two shifts were worked, not three as at Manchester, and the miners had ected up to five rings per shift.

He had intended to remark on the probability of infiltration on the length that was ied internally without the air being taken off, but the Author had already dealt with at. It was some consolation to him to know that the obstinate leaks which could not

be stopped had in fact stopped themselves after a year or so. In the Lee Valley Sewer there were one or two leaks which could not be stopped, and he hoped that within a year or so they also might have ceased.

He had wondered what purpose was served by the in-situ lining to the manhole shafts. It would seem that the shaft was sunk and presumably made watertight, and then remained in use for a considerable time before the lining was added, so there was no doubt that the lining was not needed structurally, except possibly for supporting the landings although those could be supported from the segment flanges. He would welcome the Author's comments on the value of the lining and whether he considered that a substantial saving could be made by omitting it. On the Lee Valley Sewer the manholes had been lined, except in one instance where the shaft was 13 ft 6 in. in diameter and where an excuse had been found for leaving it unlined. It had been built now for about 3 years, and still seemed to be in good trim.

Mr C. D. C. Braine (a Partner in the firm of G. B. Kershaw and Kaufman, Consulting Engineers, London) observed that he also happened to be working in the Manchester area at the moment, and he could sympathize entirely with everything Mr Murray had said about the ground there.

The Chairman had mentioned boreholes. Mr Braine had come across many strange results from boreholes, but in the Manchester area they seemed to be particularly erratic. In the early stages of the job on which he was working boreholes had been taken at reasonably close intervals, and at one particular place, where actually a manhole shaft had been located afterwards, a borehole 12 ft away from the shaft had disclosed absolutely firm clay—he called it boulder clay, but that was not the right term; it was brown and very hard, almost as good as London Clay—whereas a layer of sand about 10 ft thick was found at the bottom of the shaft. So far as was known, the whole tunnel was to be constructed in first-class clay, but subsequent experience had disclosed many cases of sand not disclosed by any of the boreholes. That raised the question whether the technique commonly adopted for putting down boreholes was not so misleading that it should be changed. He was not sure whether the misleading results sometimes obtained were the fault of the man making the boreholes, or were due to the method employed, but some of the results were so strange that one wondered whether the technique was correct.

Mention was made in the Paper of the swelling of Blue Lias lime. It was not so many years ago since it was the usual practice to use Lias lime for grouting; cement grouting came into use later. He did not recollect ever hearing before of a case where swelling of the lime had actually caused cracking of the segments. Generally speaking, the mere fact that the lime did swell was an aid to keeping out water and stopping the creep of water along the top of the tunnel.

The design of deep manholes interested him considerably. In doubtful ground the Manchester Corporation were, he thought, rather in favour of side-entrance manholes. For a side-entrance manhole so long as there was ample cover of good clay above the top of the side entrance, all was well; but in bad ground, Mr Braine considered a side-entrance manhole was difficult to construct. He would have thought that on a job where the ground was doubtful it would pay every time to have a shaft coming down straight on to the top of the tunnel—in any case the saving on the side-entrance manholes at Middleton would be very small.

With regard to the type of construction of the shafts, some of the shafts were very deep, and the ordinary concrete segment type of shaft was used, undercutting to put in rings at the bottom of the shaft as it progressed downwards. The use of a shaft of that nature in the kind of ground that had been met on the job described in the Paper, would in nearly every case lead to serious trouble and the abandonment of the shaft. He wondered whether an in-situ concrete caisson sunk the required depth in one piece might not be a better proposition. He felt sure that, provided the shafts were not too deep, it would be a much easier proposition to handle. It would be interesting to hear the Author's views on that matter.

As to the value of brickwork, he was one of those old-fashioned people who liked to see brick in a sewer, but the fact was that the cost of laying bricks in sewers today was so high that even on big sewers, with a 40-year loan, it was almost impossible to show that brickwork was economical. It looked as if for the sake of economy, one had to use concrete. It was a great pity and he entirely agreed with the Chairman on that matter, but the fact remained that the cost of brickwork might put up the cost of a job like that by £10 or £15 per yard run, which was a very heavy on-cost on the cost of the sewer.

An interesting case had been mentioned of the contractor encountering a brick sewer built previously, turning on compressed air, and then losing air through extensive leaks in the brickwork. Ordinarily, pressure tunnels were so designed that the whole pressure was taken on the lining itself, and he wondered whether in the case of the Middleton Sewer, it was not a question of literally lifting the lid off the brick sewer, when it was put under pressure, because it had not been designed to take an internal pressure of that nature. When the air was turned on, he imagined that the soffit literally moved. The internal pressure applied must have almost caused the sewer to burst, and the sewer might have been prevented from bursting by the leakages.

The question of shield or no shield was a thorny one. He knew that Mr Murray, with whom he had argued that point on sundry occasions, had said that he preferred a shield whenever there was a chance to use one, but Mr Braine knew of a firm of equal repute who did the kind of job that had been done at Middleton and who said that in no circumstances would they use a shield unless they had to. The use of a shield meant that when negotiating curves it was necessary to have very wide radii, whereas without a shield jobs could be tackled on much shorter curves. In fact, he knew of one case where the contractors had had segments specially made to get a 50-ft-rad. curve.

Mr Braine hoped that the Author in his reply, would give some costs. For instance, it would be interesting to compare the costs of straight manholes, side-entrance manholes, etc.; the cost per yard run of sewer was a bit misleading at Middleton because the tunnel was being constructed in high-pressure air one minute and in free air the next. Engineers were always in need of up-to-date costing figures for work in difficult ground, and that was a classical case where many snags had been met—where manholes had been built and then abandoned. All that enhanced the value of cost data.

Mr S. G. D. Lidstone (Engineer, John Mowlem & Co. Ltd) remarked that there was a sentence at the end of the Paper regarding the occurrence of compressed-air illness which interested him. He had been concerned recently in assisting in tendering for one or two compressed-air tunnels and, in assessing the labour costs, the implications of the revised draft regulations issued by the Ministry of Labour in 1951 in relation to work in compressed air had been considered. Probably most civil engineers had read that document, and would have noted that the emphasis was on decompression in contrast to the Institution's Code of Practice of 1936, in which recommendations were made regarding decompression times and the length of working periods in relation to the working pressure. The revised draft regulations went into a lot of detail in regard to decompression and stipulated that, where work in pressures above 18 lb/sq. in. was carried out, a decanting lock should be installed at ground level, where the men, after spending a relatively short time in the horizontal lock in the tunnel, should spend anything up to 2 hours, depending on what the working pressure in the tunnel had been. After that decanting period, they were then to be encouraged to remain on the site for anything up to another hour—all that to discourage the onset of "bends".

It would be interesting to hear whether anybody carrying out such work had attempted to adhere to the terms of the revised draft regulations and, if so, what success they had had in keeping the men on the site for the considerable period after the end of the working shift—and whether, in so doing, the onset of "bends" had been reduced to any degree. Strict adherence to the revised regulations would increase the labour cost of compressed-air tunnelling considerably, unless an appreciable decrease in the number of "bends" cases resulted.

He had been interested to hear from Mr Murray that the number of severe cases of compressed-air illness had not been too great. He presumed that the men had been locked out through the horizontal lock in the tunnel in the usual manner and that no further steps had been taken. On the other hand, the Author recorded that the number of cases had been high. One or two comments on the matter in the reply would be welcomed.

Mr F. C. Simmons (Assistant Senior Engineer, Chief Engineer's Department, London County Council) said that there were three questions that he wished to ask with regard to the curved sections of the tunnel. First, what had been the radii of the curves? Secondly, had special taper rings been used to get round the curves? If not, would the Author please indicate what method of packing on the outside of the rings had been used and whether that had been found to be successful? Thirdly, had any thin bituminous packing been tried in the circumferential joint to obtain a better water-seal?

Mr William Cathrow (formerly of Indian Railways, now retired) observed that a formation of sand, gravel, and clay made the construction of watertight sewers very difficult and there was really no solution except to use air pressure, with its high cost and difficult working conditions. There had been no boulders in the case in point and in that respect the work had not been hampered.

The pockets of sand and gravel had been formed in past geological ages by a river meandering over the flat countryside and leaving the pockets when changing course through floods, etc. The pockets were not necessarily connected one with another and it was therefore always advisable wherever possible to let the water run without causing a run of sand, to see if they were likely to dry out. It was also prudent to suspect the worst whenever sand or gravel was brought up from the test bores made before work was commenced. It might be possible to ease the situation by drying out isolated pockets, but it would not normally be possible to eliminate air pressure altogether.

Mr M. J. Tomlinson (Chief Engineer, Central Laboratory, George Wimpey & Co. Ltd) observed that the Author's account of the difficulties in construction as a result of ground conditions emphasized the need for thorough exploration by boreholes on any tunnelling project. It was important that an adequate number of boreholes should be sunk on or close to the centre-line of the work. Where compressed-air tunnelling was proposed, it was necessary to backfill the borings with well-rammed puddled clay, otherwise an unfilled borehole would form a passage for air escape.

The sections in Fig. 9 suggested that there were deep depressions in the surface of the hard clay stratum filled with sand and gravel which extended to the surface. If those depressions had been located by boring before tunnelling commenced, it might have been possible to sink deep wells at the lowest points of the water-filled depressions. Pumping from the wells might then have lowered the ground-water level below invert level, and enabled tunnelling to proceed in free air. Even though full ground-water lowering might not have been achieved, it should have been possible to use greatly reduced air pressures. Had the Author considered any form of ground-water lowering by deep wells? Possibly the presence of buildings over the line of the sewer might have influenced a decision against such methods.

The Author, in reply to the Chairman's questions about ground conditions, and the Direct Works Section, said it had not been intended that the Section would necessarily construct the full length of the sewer. If the Direct Works Section had been able to complete the first section they would probably have continued with the second section; however, when it became evident that compressed air was required the completion of the sewer had automatically been let to contract.

As mentioned in the Paper a small number of boreholes—the Author believed six—had been taken for the second section. For some reason, probably because the line of sewer mainly followed the roadways, most of the boreholes had been put down some distance

from the sewer, and their value was correspondingly limited. The borehole data provided were very meagre, but the Author doubted whether a more extensive survey would have been much more helpful, unless it had been on a scale to give an almost complete picture of ground conditions; the cost of such a survey would, almost certainly, have been prohibitive. Also, the levels at the ends of the sewer were fixed by existing conditions, and since the resulting gradient was only about 1 in 400, intermediate levels could not have been varied to any great extent to get into good ground. Checking borehole information in doubtful ground by trial pits was useful, but was usually impracticable at depths exceeding about 20 ft.

The Author emphasized that where the centre-lines and the centre of the barrel did not coincide that resulted from the sewer being off-centre, not to the centre-lines being out. The centre-lines were correct, but that had not been established until tunnelling on the first section had been completed. At the time the survey operations were carried out their accuracy seemed doubtful and could not be taken for granted.

The cost of the work did exceed the original fixed-price tenders, which were between £250,000 and £300,000.

In reply to Mr Murray, whilst the Author did not discount the suggestion that the pressure of 32 lb/sq. in. might have been connected with the very fine nature of the sand, he thought it significant that only in the length where that pressure was used had difficulty been experienced in sealing off subsequent percolation of ground-water. The Author felt that they had, in fact, had about 70 ft head of water above them; alternatively, the pressure at the face might have been a few lb/sq. in. less than the pressure in the air lock, owing to the large air losses which took place through cracked segments and uncaulked nvert joints.

The Author agreed that, when segments were subsequently to be lined, wooden plugs would provide an answer to the problem of concrete plugs blowing out under air pressure.

Since two miners had been employed on all faces on the connecting sewer, it had not been possible to establish from actual costs that it was economical to construct a sewer a foot or so larger than the size required, to get two miners in the face. Such comparisons were always difficult unless such factors as ground conditions, length of haul, working capacities of the miners, etc., were similar in both cases; a misleading conclusion could be drawn if that were not so. On the Lee Valley Sewer those factors had probably operated n favour of the 4-ft-6-in.-dia. sewer, since Mr Hughes had mentioned that the best working conditions were found in that sewer.

The Author shared Mr Hughes's doubts on the need to provide an in-situ lining to nanhole shafts. In considering the question the Corporation had been influenced by the act that, despite extensive caulking, the shafts were not completely watertight. The ining did help to provide a more positive support for the landings, and on the Middleton Connecting Sewer the landings, in turn, supported drop shafts. Lining the shafts had een chosen but it might have been a waste of money.

The Author agreed with Mr Braine's remarks on boreholes. Too many could not be sed, but if all the boreholes necessary to give an adequate picture of ground conditions n an area such as North Manchester were put down, they would probably cost almost as nuch as the job! One had to draw the line somewhere.

There was no doubt that borehole data were often misleading, particularly in ground ontaining water-bearing strata, where the water might have been allowed to follow the orehole into dry ground below. The best insurance against inaccurate information was o entrust the ground survey only to recognized experts, who would submit a comprehensive report of their findings together with details of the techniques adopted.

The ground behind the segments, fractured after grouting with Lias lime, was an xtremely hard clay. Had the ground been softer it might have yielded slightly and the egments might not then have cracked. Freedom from that trouble with cast-iron egments was probably a matter of relative compressive strengths.

The Author thought that an in-situ concrete caisson sunk in one piece would not possess ny advantage over a caisson built up ring-by-ring as sinking proceeded. He could, owever, think of a number of disadvantages; e.g., the caisson would be difficult to control

and might fracture if it got out of hand, or it might refuse to go down far enough, which would necessitate cutting off the top and might involve a difficult change-over to underpinning methods at the bottom. There would also be a delay for manufacture and subsequent curing, compared with precast segments. That would not necessarily be a disadvantage but it might be important in cases, such as at manhole 4 on the second section, where the contractor had to get the shaft down as quickly as possible to minimize interference with road traffic.

Some anxiety had been felt that the existing sewers might burst when subjected to air pressure but no difficulty was experienced; the only damage was a crack in one of the walls of manhole 4.

The Author felt that it would be unwise to enlarge on the details of costs given in the Paper, since the wide variations in ground conditions, size of manhole shafts, alternation of work from free to compressed air, etc., meant that cost data could be misleading unless all those factors were taken into account.

In reply to Mr Lidstone, the Author observed that although the draft regulations became available during the course of the contract they had worked to the recommendations and decompression times given in the Institution's Code. Mr Lidstone had quoted him as saying that the number of cases of "bends" had been high; he had in fact said "numerous", which was not quite the same. The Author had unfortunately mislaid his records of compressed-air illness on the contract, but he thought that the number of cases had been rather higher than mentioned by Mr Murray. With one exception—and in that case it was doubtful if the workman had really had the "bends"—all cases had occurred at pressures higher than 18 lb/sq. in.

Mr Simmonds had asked about the radii of the curves. The contract had called for curves of 200-ft radius, but that had been increased to 300-ft on the shield drive. The method of packing had not been quite as he would have liked to have seen it. Ideally, one would expect to have a packing ring tapering in thickness from about 1 in. on the outside of the curve to, say, $\frac{1}{4}$ in. on the inside; the ring would be inserted in the circumferential joint, the segments bolted up, and everything would be neat and tight. However, it did not always work out that way, since the shield, although it successfully negotiated the curve did not necessarily keep exactly on line, and the amount of packing required was not, therefore, the same at every joint. As a result the packing had been built up of $\frac{1}{4}$-in.-thick slats of wood, four or five pieces being used on the wide side, one piece on the narrow side, and an intermediate number in-between. Fortunately the curves at manholes 3 and 4 had been mainly in dry ground, since it was more difficult to caulk on a curve; a less satisfactory groove was obtained.

Bituminous packings had not been used on the circumferential joints, and he doubted if they would materially assist in obtaining a better water-seal.

In reply to Mr Tomlinson, the possibility of ground-water lowering by deep wells had not been considered on the second section. The idea was attractive, but the Author doubted whether the method would have been economical at the depth concerned, which was generally about 100 ft.

Well-points had been used by the Direct Works Section adjacent to manhole 4 on the first section, where the depth from ground to invert level was about 35 ft. He believed the results had been disappointing, but that may have been owing to the equipment and technique employed, which was probably less efficient than those now in use.

STRUCTURAL AND BUILDING DIVISION MEETING

10 May, 1956

Mr Ralph Freeman, Member, Chairman of the Division, in the Chair

The following Paper was presented for discussion and, on the motion of the 'hairman, the thanks of the Division were accorded to the Authors.

Structural Paper No. 50

THE ALLOWABLE SETTLEMENTS OF BUILDINGS

by

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and

Donald Hugh MacDonald, Ph.D., M.E.I.C.

SYNOPSIS

A knowledge of the allowable settlements of buildings is necessary for rational founda-on design. The usual method of structural analysis can give only a partial solution- owever, and recourse must therefore be made to observations on actual buildings. The aper summarizes the results of a survey of existing data on ninety-eight buildings, of hich fifty-eight have suffered no damage and forty have been damaged, in varying egree, as a consequence of settlements. From this survey it has been possible to establish entative values for damage limits in terms of angular distortion and, with less accuracy, terms of maximum and differential settlements. Allowable settlements and distortions re suggested as a basis for design. The available data refer to load-bearing wall struc-ires and to steel or reinforced concrete frame buildings with panel walls of brick or milar construction.

INTRODUCTION

r is possible to predict the maximum settlements of a building, with some degree f reliability, from field or laboratory tests on the underlying soil. Settlements are sually most important on clays, and with these soils calculation methods usually volve an error of not more than 50%, as proved by comparisons between observa-on and calculation.[1] On sandy soils methods of prediction are nearly always less tisfactory but, fortunately, settlements on sands are in most cases considerably naller than on clays.

No matter how accurate a settlement analysis may be, however, it is of limited ractical value if the designer is not aware of the amount of settlement which can allowed for the particular building under consideration. In other words, a know-dge of allowable settlement is as important as the ability to carry out a settlement lculation. Yet considerably less attention has been given to the former question.

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[1] The references are given on p. 761.

One reason for this lack of balance in foundation research may well be the fact that it is very difficult to compute the allowable settlements of a building by the usual structural methods. Consequently it is necessary to use a purely observational approach to the problem. But such an approach, which is to some extent statistical, can be adopted only when a reasonably large number of case records are available. It is only during the past 15 years that a sufficient number of records have been published for an observation study to be made. But the data are restricted almost entirely to load-bearing wall structures and what may be called traditional steel or reinforced concrete frame buildings, comprising stanchions and beams, with brick panel walls (sometimes with stone facing slabs) and brick or block partitions. Hardly any data, and none of critical value, so far seem to be available for frame buildings with modern forms of curtain walling and dry-construction partitions. These limitations must be borne in mind while reading the present Paper.

Indeed, even for the more traditional type of buildings, the Authors have been able to achieve no more than a preliminary survey of the subject of allowable settlements.

Stresses in building frames

Conventional structural design of a framed building involves computation of stresses in the stanchions and beams, for the condition of no differential settlement. The same techniques of analysis can, however, also be applied to calculating the stress changes that occur in the frame when differential settlements take place. For example, Meyerhof[2] has analysed a five-storey three-bay reinforced concrete frame and he found that a differential settlement $\delta = 0{\cdot}315$ in. in a span of $l = 25$ ft (and hence an angular distortion $\delta/l = 1/950$) caused an increase of 74% in the bending moment in the beam which was subjected to the largest moment prior to settlement. Now, an increase of this order, in a beam already at its working stress, would be expected to cause some damage in the form of cracking; but from data given later in the Paper it is certain that an angular distortion of 1/950 does not, in fact, cause any cracking. Moreover, appreciably greater distortions are known to cause no cracking in the frames of actual buildings.

There are at least three possible explanations of this anomaly. First, the live loads assumed in design are necessarily conservative compared with the actual average loads in buildings. Secondly, as a consequence of these conservative assumptions of live load, but also as a consequence of the composite action of frame casing, floors, and wall panels, the stresses and deflexions in the frame of a building may be lower than the design values; and thirdly, the additional stresses in the frame caused by differential settlements may also be lower than those calculated by the usual form of analysis which considers the statics of the frame alone.

With regard to the average live loads in buildings there is ample evidence that they are less than the values used in design.[3, 4] In relation to the second possibility, namely, that the actual stresses in the frames of buildings, without differential settlement, are lower than the usually accepted working stresses, the evidence is scanty. Measurements have been made,[5] however, in the new Government offices at Whitehall Gardens, and show that when the building was completed, in 1951, the maximum stresses in the beams were between one-quarter and three-quarters of the calculated values and, moreover, the additional stresses due to applied live loads were in most cases inappreciable compared with those which would be anticipated from calculation. Somewhat similar results were obtained [6] when test loads were

pplied to the steel frame of the Cumberland Hotel, London, in 1933. With the rame uncased the beam stresses were about 10% less than the calculated values, et with the frame cased and the floors laid the stresses per unit applied load were ound to be only 40 to 50% of those measured with the frame bare. The situation egarding stanchion stresses appears to be rather different, as might be expected. 'hus in the Geological Museum, Kensington, measurements made in 1931 show [7] hat, with the frame cased and the floors laid, the direct stresses in the stanchions ere in close agreement with calculations based on a careful estimate of the actual oading. The bending stresses, however, were greater than those estimated by the onventional design method of that time. Allowing for the usual conservative live-oad assumptions it might be said, therefore, that the actual stanchion stresses ould differ by only a small amount from the allowable values.

The available data on reinforced concrete buildings indicate a generally similar attern to that for steel-frame buildings. The maximum measured stresses, for xample, at test loadings equal to the design live load, were found to be about 50 o 75% of the calculated stresses in the beams of two American warehouses [8] investi-ated in 1911. In a third warehouse of the same period, with flat-slab floor con-truction, the stresses in the reinforcement did not exceed one-half the allowable alues, although the concrete stress appeared to be more nearly equal to that per-iitted by the contemporary building ordinances.[9] Tests in 1952 on the floor ystems of a modern reinforced concrete building in Johannesburg, completed in 942, also indicated the somewhat conservative nature of the conventional design iethods.[10] Little field data exist on the stresses in columns in reinforced concrete uildings, but there seems to be no reason why the conclusion previously mentioned or steel stanchions is not equally applicable, at least in a general manner.

Turning now to the question of stresses arising from differential settlements, the uthors are aware of only two records where a comparison has been made between bservations and the calculated stresses set up in the steel framework of actual build-igs due to a known differential movement. The first record concerns the Charity Iospital, New Orleans,[11] where calculations based on the measured settlements, and iaking some allowance for the rigidity of the reinforced concrete floors, indicated iat the maximum stress in a particular stanchion was somewhat in excess of the ield point of the steel. No stress measurements were made in the frame but it was iferred from deflexions of this stanchion that the stress was probably not far below ie value estimated in the computation. The second record [12] concerns the building i Whitehall Gardens to which reference has already been made. The results here, owever, were appreciably different from that reported for the stanchion in the harity Hospital. In the Whitehall Gardens offices the bending stresses set up by ifferential settlement were measured in a number of beams and stanchions, and ere found to be of the order of 30% of those computed. This figure cannot be iterpreted too literally, since it is not easy to quote a single value for the ratio of alculated to observed stress, owing to the fact that the analysis implies simple gid-frame action whereas the measurements indicate a more complex structural ehaviour, brought about by the composite action of the panel walls, floors, and ncased frame.

The general conclusion from these cases is, therefore, that it is difficult to predict n allowable differential settlement from stress calculations in the framework of uildings.

Cracking in panels

There is, however, a further point of considerable relevance. Field evidence on frame buildings of the traditional type shows that allowable settlement is governed more by the avoidance of cracking in the panels and finishes, than by the over-stressing of the stanchions and beams; although it must be mentioned that diagonal bracing seems in some cases to be even more sensitive to differential settlement than are panel walls.

Settlement damage in frame buildings can be divided into the following three categories:—

(1) "Structural", involving only the frame, i.e., stanchions and beams.
(2) "Architectural", involving only the panel walls, floors, or finishes.
(3) Combined structural and architectural damage.

The records of twenty-five damaged frame buildings have been studied (see Table 7) and the incidence of damage occurring under each of these categories is given in Table 1, for two principal types of building:—

(*a*) "mill" type, such as warehouses, factories, etc., where there are few, if any, internal walls and little in the form of finishings; and
(*b*) "office" type, such as office blocks, flats, hotels, hospitals, etc.

From Table 1 it is evident that damage to the columns and beams unaccompanied by any "architectural" damage is rare; only two cases are known to the Authors and those are both light unclad steel-frame buildings. In contrast, twelve cases are known of damage solely to the walls (interior or exterior), floors, and finishings; moreover, of the thirteen cases involving structural damage, eleven were accompanied by cracking in the walls, etc.

TABLE 1.—FREQUENCY OF OCCURRENCE OF VARIOUS CATEGORIES OF DAMAGE IN FRAME BUILDINGS

Damage	Number of damaged buildings	
	(*a*) "mill" type	(*b*) "office" type
(1) Structural only	2	0
(2) Architectural only (including tilting)	4	8
(3) Architectural and structural	4	7
Total	10	15

The fact that architectural damage, such as cracking of wall panels, is likely to occur at smaller distortions of the building than is structural damage, can also be deduced from tests recently carried out at the Building Research Station.* In these tests a shearing force was applied to a full-scale encased steel frame, both open and filled with various panel walls consisting of $4\frac{1}{2}$-in. brickwork, 3-in. hollow-tile blocks or 3-in. clinker blocks. With the open frame, cracking first became visible at an

* The tests are briefly described by Thomas.[13] The Authors are indebted to Mr L. G. Simms and Dr F. G. Thomas for the data, which are quoted by permission of the Director of Building Research, Department of Scientific and Industrial Research. These have been used in connexion with foundation studies by Ward[27] and Meyerhof.[12]

ıgular distortion $\delta/l = 1/120$. When a panel was in position, however, cracking rst appeared in the panel at distortions ranging from 1/450 to 1/300 for the different anels mentioned above. Hence the limiting distortion for the panels is about one-iird of that for the open frame. But the tests are also of interest because the actual alues of the distortion causing cracking in the panels are similar to the damage mit of δ/l as found from the survey described in the Paper.

These tests, coupled with the field evidence in Table 1, show that the allowable ettlement of a frame building is, in the majority of cases, determined by the ehaviour of the panels and other non-structural elements. But the limits of dis-ortion permissible in such elements are not, in general, susceptible to calculation. herefore, except in a few instances, allowable settlements can be found only from oservations on actual buildings or, possibly, from full-scale laboratory tests.

oad-bearing walls

In the same way it is difficult to calculate the limiting distortion of load-bearing alls, with their window and door openings, the effects of creep, and other uncer-ıinties. Here again, recourse must be made to field evidence on actual buildings.

isual limits

Finally, in this matter, it may be noted that limits for allowable settlements can e imposed by purely visual effects; notably the tilt or lean of a tall building hich may well reach an unacceptable magnitude (aesthetically or psychologically) efore any cracking has occurred. It is recognized that the limit of tilting is very nportant in the design of foundations for high buildings, but research is still in rogress into the data concerning this problem, and the present Paper deals ex-usively with allowable vertical settlements, from the point of view of avoiding racking and other effects consequent upon distortions in a building. There is. owever, some evidence that tilting can be easily seen when it amounts to about 1/250, nother visual limit is imposed when a building is erected immediately adjacent to n existing structure with which there is some architectural alignment, such as a ring course or parapet.

Nature of the criteria of allowable settlements

The settlement characteristic causing cracking is probably the radius of curvature. ut a characteristic which is more readily evaluated, and which is only slightly less gical, is the angular distortion; this is conveniently expressed by the ratio of the fferential settlement δ and the distance l between two points (Fig. 1). Ideally ere would be available a body of performance records relating δ/l to the presence absence of damage at any particular place in a building. In fact, however, the ımber of buildings for which such evidence exists is small. Yet by assuming that e worst damage occurs where δ/l is maximum, it has been possible, in addition, obtain a considerable amount of indirect evidence. From all these field data on acking in buildings a limiting value of $\delta/l = 1/300$ has been obtained; as will be ıown later.

To this extent, therefore, the allowable settlement of the types of buildings studied this Paper can be said to have been established, at least in a preliminary manner. ıe usefulness of this criterion, however, is rather limited on account of the uncer-inties in calculating the values of δ/l which may be expected to occur in any given se. In practice, if the settlements are caused essentially by the consolidation of a

deep-seated layer (or layers) of clay, and the foundation rests on sand or gravel above the clay, the settlement pattern can be calculated with some confidence. But if the foundation bears directly on clay, and still more if it rests on sand with no clay underneath, the prediction of settlement patterns is complicated by the inevitable local variations in the soil, which control the distribution of settlements to an important degree. Yet, even in such cases, the maximum settlements can usually be

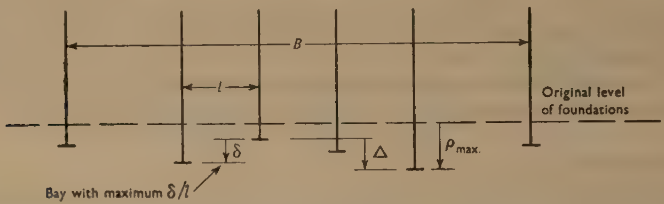


FIG. 1.—DIAGRAM ILLUSTRATING THE DEFINITIONS OF MAXIMUM ANGULAR DISTORTION δ/l, MAXIMUM SETTLEMENT $\rho_{max.}$, AND GREATEST DIFFERENTIAL SETTLEMENT Δ FOR A BUILDING WITH NO TILT

calculated with some measure of accuracy, as also can the greatest differential settlements. Consequently it is desirable to see if, for any particular type of building and foundation conditions, a statistical correlation can be found between the greatest value of δ/l (the logical criterion of damage) and the maximum settlement $\rho_{max.}$ (the most readily computed quantity in a settlement analysis). Also, for similar reasons, the possibility of a correlation between δ/l and the greatest differential settlement Δ has been investigated (see Fig. 1 for diagram illustrating δ/l, $\rho_{max.}$, and Δ).

Fortunately, for foundations bearing directly on clay or sand, approximate correlations of this type have been found to exist. Hence the allowable settlement of buildings can be given not only in terms of δ/l but also (at least as a guide) in terms of the more practical criteria of maximum settlement and greatest differential settlement.

Therefore, the consideration of the field evidence falls under three main headings; first the damage limit of the angular distortion δ/l, secondly the relations between δ/l and the maximum settlement and the greatest differential settlement, thirdly the establishment of damage limits in terms of these latter two settlements.

DATA ON ANGULAR DISTORTION

The values of angular distortion given under this heading have been obtained from settlement contours or profiles, with the component due solely to tilting eliminated. (See Appendix I for examples.)

Direct evidence

Data on the presence or absence of damage at five particular points in a building and the corresponding values of δ/l, were obtained from a single-storey steel-frame building [71] * at Kippen, near Stirling. This had external brick panel walls, and

* Numbers in square brackets relate to the buildings, a complete list of which is given in Table 7. References to the literature describing these buildings will be found in Appendix III.

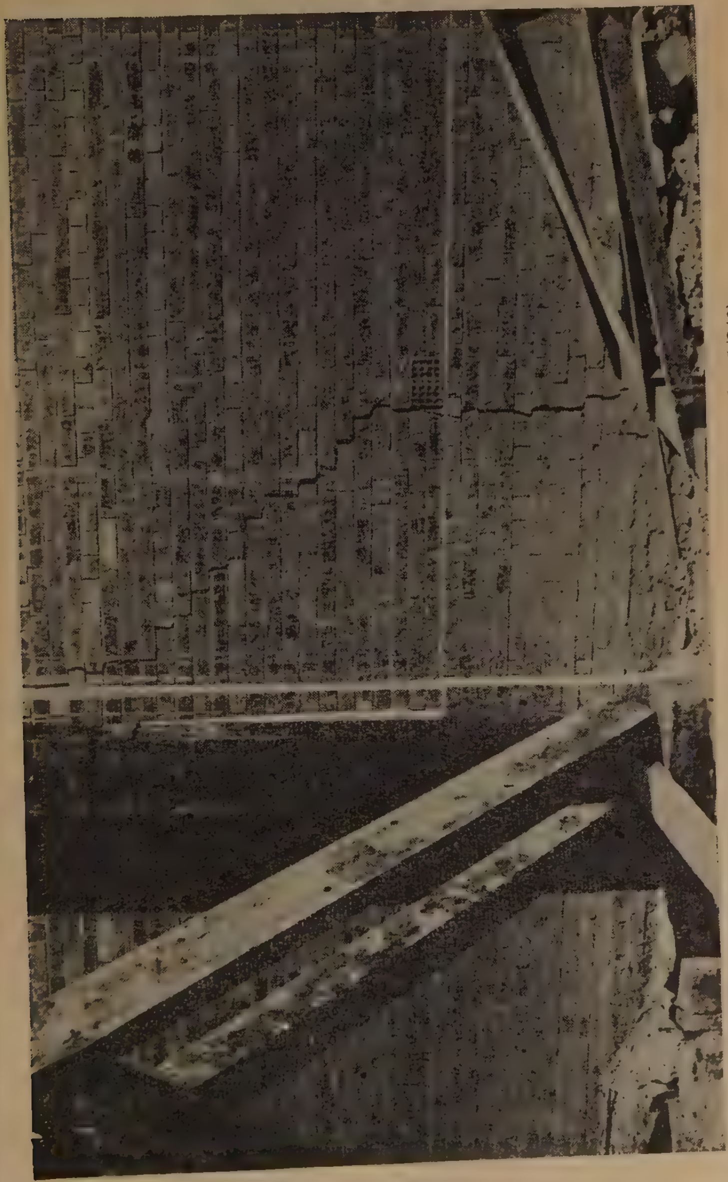


Fig. 2.—Crack in brick panel of building [71] ($\delta/l = 1/240$)
Failure during construction

TABLE 2.—DIRECT EVIDENCE ON FRAME BUILDINGS WITH PANEL WALLS

KEY:—C denotes clay, S sand, O "office" type, M "mill" type, F footings, R raft, and P pile

	Building	Ref. No.	Soil type	Building type	Foundn	Width of building: feet	Angular distortion: $\frac{\delta}{l}$	Max. settlement, ρ max.: inches	Ratio $\frac{\delta/l}{\rho\text{max.}}$: inches^{-1}	Greatest differential settlement: inches
No cracking in panels	Apartment bldg, Ottawa	2	C	O	R	69	1 : 3,800 *	0·28	1 : 1,060	0·23
	Fire Testing Station, Elstree	12	C	M	F	36	1 : 2,800 *	0·71	1 : 1,980	0·45
	Mt Sinai Hospital, Toronto	5	C	O	R	48	1 : 1,650 *	0·45	1 : 740	0·40
	Kippen warehouse, Stirling	71	C	M	F	100	1 : 720	—	—	—
	Whitehall Gdns, London	32	C	O	R	100	1 : 480 *	1·85	1 : 890	0·64
Cracking in panels	Kippen warehouse, Stirling	71	C	M	F	100	1 : 290	—	—	—
	,, ,, ,,	71	C	M	F	100	1 : 240	—	—	—
	Charity Hospt, New Orleans	74	C	O	R P	110	1 : 100 *	11·0	1 : 1,100	9·0
Structural damage	Charity Hospt, New Orleans	74	C	O	R P	110	1 : 150	—	—	—
	Kippen warehouse, Stirling	71	C	M	F	100	1 : 110	—	—	—
	,, ,, ,,	71	C	M	F	100	1 : 80 *	9·8	1 : 780	9·8

* These values are maximum for buildings.

TABLE 3.—DIRECT EVDENCE ON LOAD-BEARING BRICK-WALL BUILDINGS

KEY as for Table 2

	Building	Ref. No.	Soil type	Building type	Foundn	Width of Building: feet	Angular distortion: $\frac{\delta}{l}$	Max. settlement, ρmax.: inches	Ratio $\frac{\delta/l}{\rho\text{max.}}$: inches^{-1}	Greatest differential settlement: inches
No cracking	Brick building (5), Vienna	6	S	O	F P	36	1 : 950 *	0·50	1 : 480	0·34
	Brick building (20), Vienna	15	S	O	F P	50	1 : 900 *	0·88	1 : 790	0·59
	Municipality bldg, Minieh	46	C	O	R	60	1 : 720 *	1·73	1 : 1,250	0·90
	Building C	41	S	O	F	—	1 : 500 †	2·76	—	1·03
	Building A	33	Fill	O	R	—	1 : 380 †	2·11	—	1·40
	Building D	35	C	O	F	—	1 : 285 †	2·34	—	0·47
Cracking	Five-storey building (4)	57	Fill	O	R	75	1 : 270	—	—	—
	” ”	57	Fill	O	R	75	1 : 120 *	5·8	1 : 700	5·6

* These values are maximum for buildings. † Probably not max. δ/l for whole building.

TABLE 4.—INDIRECT EVIDENCE ON FRAME BUILDINGS WITH PANEL WALLS

KEY:—C denotes clay, S sand, O "office" type, M "mill" type, F footings, R raft, P pile or pier, and B box

	Building	Ref. No.	Soil type	Building type	Foundn	Width of building: feet	Angular distortion: $\frac{\delta}{l}$	Max. settlement, $\rho_{max.}$: inches	Ratio $\frac{\delta/l}{\rho_{max.}}$: inches^{-1}	Greatest differential settlement: inches
No damage reported	Novo Mundo, São Paulo	3	S	O	F	105	1 : 2,900	0·36	1 : 1,040	0·28
	Building XI, Egypt	19	C	O	F P	45	1 : 1,800	0·75	1 : 1,350	0·30
	Azevedo e Villares, São Paulo	16	S	O	F P	47	1 : 1,650	0·80	1 : 1,320	0·30
	Building III, Egypt	13	C	M	F	92	1 : 1,600	0·59	1 : 940	0·38
	Riscala, São Paulo	4	S	O	F	100	1 : 1,250	0·35	1 : 440	0·20
	Building V, Egypt	37	C	O	F P	50	1 : 1,100	1·38	1 : 1,520	0·38
	Pacific G & E, San Francisco	45	C	O	F P	140	1 : 1,050	3·1	1 : 3,260	3·1
	San Jacinto Monument, Texas	53	C	O	R	124	1 : 920	5·1	1 : 4,680	0·6
	Banco do Estado, São Paulo	17	S	O	R P	80	1 : 900	0·80	1 : 720	0·3
	Thomas Edison Bldg, São Paulo	10	S	O	R	65	1 : 850	0·60	1 : 510	0·49
	C.B.I. Esplanada, São Paulo	21	S	O	F	115	1 : 760	1·05	1 : 800	0·43
	Building X, Egypt	27	S	O	F P	64	1 : 680	1·18	1 : 810	0·38
	Hayden Library, Boston	20	C	O	F P	190	1 : 670	1·0	1 : 670	0·75
	New England Mutual, Boston	34	C	O	B	194	1 : 650	1·80	1 : 1,170	1·2
	Ohio Bell Telephone, Cleveland	26	C	O	B	155	1 : 650	1·63	1 : 1,060	0·94
	Nonoalco Power, Mexico City	76	C	M	B	68	1 : 620	>10·5	1 : 6,500	8·5
	Liberty Mutual, Boston	36	C	O	F P	127	1 : 550	1·85	1 : 1,020	1·02
	Hotel São Paulo	11	S	O	F P	85	1 : 500	0·58	1 : 290	0·32
	Ipiranga, São Paulo	24	C	O	F	116	1 : 410	1·20	1 : 490	0·75
	Shell Oil, San Francisco	65	C	O	F P	118	1 : 370	7·1	1 : 2,620	1·92
	Standard Oil, San Francisco	75	C	O	R	138	1 : 270	9·8	1 : 2,640	3·9
	Locomotive shed, Kerava	88	C	M	R	62	1 : 260	16·7 *	1 : 4,340 *	4·5
	Building VII, Egypt	25	S	O	F P	70	1 : 180	1·14	1 : 210	1·10

Wall or floor cracking	National Theatre, Mexico City	98	C	O	R	250	1 : 270 †	60	—	19
	Admin. Bldg, M.I.T., Boston	67	C	O	F P	140	1 : 200	8·2	1 : 1,640	2·2
	Apartment Building, Chicago ‡	93	C	O	R	120	1 : 170	21	1 : 3,560	10·5
Structural damage and wall cracking	Gare Maritime, Le Hâvre	68	C	O	F P	150	1 : 65	8·5	1 : 550	8·5
	U.S. Navy Bldg No. 76, Phil.	87	C	M	F P	180	1 : 55	24	1 : 1,320	14
	Factory Building, Wembley	82	Fill	M	F P	100	1 : 50	12·2	1 : 610	8·0
	Retort House, Weston-s.-Mare	85	C	M	R	65	1 : 50	15	1 : 750	11·5
	Masonic Temple, Chicago §	77	C	O	F	135	1 : 35	10·9	1 : 380	6·9
	Auditorium, Chicago	95	C	O	F	176	1 : 30	30·5	1 : 920	30·0
	Jurgens Oil Mill, Zwijndrecht	96	S	M	F P	55	1 : 18	25	1 : 450	25

* Settlement caused largely by general filling
‡ Tilting and probably some panel cracking
† Certainly not max. δ/l for whole building
§ Possibly only panel cracking

TABLE 5.—INDIRECT EVIDENCE ON LOAD-BEARING BRICK-WALL BUILDINGS

KEY:—C denotes clay, O "office" type, and F footings

	Building	Ref. No.	Soil type	Building type	Foundn	Width of building: feet	Angular distortion: $\frac{\delta}{l}$	Max. settlement (ρmax.): inches	Ratio $\frac{\delta/l}{\rho\text{max.}}$: inches^{-1}	Greatest differential settlement: inches
Damage	Old Board of Trade, Chicago	73	C	O	F	183	1 : 105	10·2	1 : 1,070	5·6
	Monadnock Block, Chicago	91	C	O	F	88	1 : 90	17·4	1 : 1,560	5·3
	Post Office, Bregenz	61	C	O	F	63	1 : 30	32	1 : 960	22·5
	National Museum, Ottawa	90	C	O	F	65	1 : 30	19·2	1 : 580	19·2
	Mixed Law Courts, Cairo	64	—	O	F	300	1 : 25	9·5	1 : 240	8·5

it suffered large settlements owing to a foundation failure during construction. The results of this study may be summarized as follows:—

Severe distortions of steel frame	$\delta/l = 1/110, 1/80$
Cracking in panel walls	$\delta/l = 1/290, 1/240$
No cracking in panel walls	$\delta/l = 1/720$

The cracking in a wall panel of this building, in which the differential settlement $\delta = 1\frac{1}{2}$ in. for a span $l = 30$ ft (i.e., $\delta/l = 1/240$) is shown in Fig. 2, where the temporary shoring for the underpinning operations can also be seen.

In the Charity Hospital, New Orleans, it is recorded [74c] that severe cracking occurred in an exterior wall panel where $\delta = 2\frac{1}{4}$ in. in a span of 20 ft ($\delta/l = 1/100$). There is, of course, no suggestion that cracking did not occur elsewhere at smaller values of δ/l. In the same building the maximum stresses in an external stanchion were considered to have reached the yield point at a value of $\delta/l = 1/150$.

Direct evidence, of a rather different nature, can be obtained from four frame buildings for which the maximum value of δ/l is known and, at the same time, it is also specifically stated that no settlement damage occurred anywhere in the building. These four cases are included in Table 2, which summarizes the direct evidence available from the various frame buildings discussed above.

From Table 2 it can be seen that the damage limit for the angular distortion of panel walls is between 1/300 and 1/450, and that the limit for structural damage in the frame is 1/150. It must be emphasized, however, that these facts concern buildings of the usual beam and stanchion construction. Observations * on a group of industrial buildings in Canada have shown that buckling can occur in steel diagonal bracing at angular distortions as small as 1/600. This fact underlines the restrictions of the present Paper, mentioned earlier.

In a valuable Paper by Terzaghi,[14] information is given on six buildings with load-bearing brick walls. From those data, and from one further case [46], the results in Table 3 may be deduced; the value of δ being taken over a length l of not less than 15 ft. The limiting angular distortion here appears to be established at a value of about 1/280.

Indirect evidence

There are a number of buildings concerning which a sufficient amount of information exists on the settlements for the maximum value of δ/l to be obtained, and concerning which there is information of one or other of two classes:—

(1) that settlement damage has occurred, although the exact position of occurrence of this damage is not stated; or
(2) that, so far as is known, no settlement damage has occurred.

The cases of class (2) are obviously similar to the four buildings already mentioned above under the heading "Direct evidence"; the only difference being that in the latter buildings it is definitely stated that no damage occurred whereas, in the cases at present under consideration, this is only an inference from the fact that no damage was reported. Most of these records therefore provide useful negative evidence. The cases of class (1) are individually of very limited value, since damage may well have occurred not only where δ/l is a maximum, but also at other places in the building where δ/l is appreciably lower than the maximum. Nevertheless, whe

* The Authors are indebted to Dean H. M. Hardy, University of Alberta, and Mr C. I Ripley for this information.

nsidered as a group, the cases of class (1) are useful in so far as the set of results nds to indicate a lower limit for the angular distortion causing damage.

The real merit of this indirect evidence is, however, in the combination of evidence classes (1) and (2), from which an indication can be obtained of the limit of δ/l; eater values leading to damage and smaller values causing no damage. The idence is set out in Tables 4 and 5.

From Table 4 it might be assumed that no damage will occur, at least in some ıme buildings, at distortions even as large as about 1 : 200. This may be true, ıt the indirect nature of the evidence should be recalled, and also it will be seen at cracking occurred in two buildings with maximum angular distortions of 200 and 1 : 270. Therefore, in general, the conclusion from this evidence is that limit of about $\delta/l = 1/300$ cannot safely be exceeded; this conclusion is in agree- ent with the deduction made from the direct evidence previously given in Table 2. ıe information in Table 5 on load-bearing wall buildings is too scanty to permit y definite conclusion to be made, although there is no conflict with the comparable rect evidence in Table 3.

For the sake of completeness, the type of building and foundation have been ven in the foregoing Tables, but there is no indication of any relation between these tors and the limiting value of δ/l causing damage.

An attempt has been made to classify the soil profile as predominantly "sand" ıd gravel), or "clay" (and silt), or "filling". As will be seen later, the type of l has an important influence on the relation between settlement and angular tortion. But it is not a major factor in the relation between angular distortion d the absence or presence of damage; although somewhat greater distortions may safe when associated with long-term settlements of clay, owing to the small rates strain which allow some relief of stress in the building materials.

mmary of evidence

To illustrate the evidence concerning settlement damage and angular distortion, data in Tables 2 to 5 have been expressed graphically in Fig. 3. This shows arly that it would not be justifiable to take a value of more than 1/300 as the mage limit. There are, in fact, four undamaged buildings with angular distortions ater than 1/300, but in three of these [35], [75], and [88] δ/l does not exceed 1/260 l, therefore, they are not in any material disagreement with the suggested limit. e only case differing substantially is building [25] with $\delta/l = 180$; but it is hardly prising to find an occasional exception. The very sharp change in behaviour wn in Fig. 3, between buildings with maximum angular distortions of less than 00 and more than 1/250 is, indeed, probably fortuitous.

Two of the panel walls tested at the Building Research Station cracked at dis- tions of 1/350 and 1/450, i.e., at values of δ/l smaller than the suggested limit. ce again the differences are not great, but they can probably be explained by the ısually strong mortar used in the test panels, and also by the rate of strain in the oratory tests being necessarily so much greater than that occurring in practice.

Therefore it is considered that an angular distortion of 1/300 can be accepted as a sonable value for the limit, above which cracking is likely to occur in load-bearing lls and in the panel walls and masonry facing of frame buildings. For a typical width of 20 ft this angular distortion is developed by a differential vertical vement of $\frac{3}{4}$ in. between adjacent columns or cross-walls.* Moreover, if the

* Equivalent to a radius of curvature of 3,000 ft.

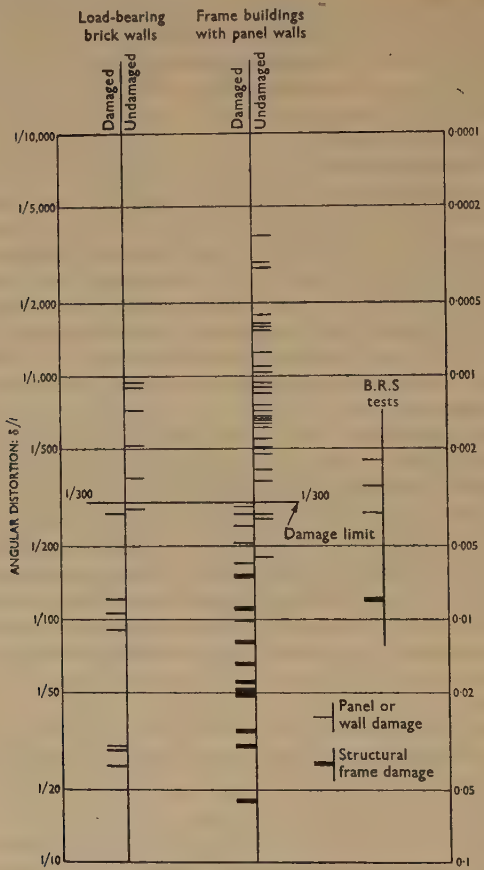


Fig. 3.—Field evidence on damage (related to angular distortion)

distortion amounts to 1/150 (equal to a differential movement of $1\frac{1}{2}$ in. in a span of 20 ft), it is likely that structural damage will occur in the beams or stanchions.

Relations between the settlement criteria

Maximum settlement and angular distortion

For reasons given earlier it is desirable, if possible, to establish correlations between the maximum settlement $\rho_{\max.}$ and the greatest angular distortion δ/l. The basis for such correlations is simply that the larger the settlement the more probable it is that the angular distortion will also be greater. But it is obvious that there cannot be one unique correlation independent of the type of foundation and the soil conditions.

At the outset it must be recognized that those cases where settlement is due to a deep-seated layer of clay cannot properly be considered in this connexion since the relation between δ/l and $\rho_{\max.}$ varies not only in the manner just indicated but also

ith the depth to the layer and its thickness. Of the fifty buildings for which ata exist (Tables 2 to 5) on angular distortion, there are five belonging to this ıtegory; [24], [45], and [61] on footings, and [74] and [75] on rafts. Another con-ition which cannot be considered in this aspect of the problem is exemplified by uilding [88] where the large maximum settlement of about 17 in. is due almost ıtirely to a general filling of compacted gravel 10 ft thick, covering a wide area and verlying 35 ft of soft clay. The light single-storey building itself contributes little this settlement and hence the angular distortions bear almost no relation to the aximum settlement. (Similar conditions occur at the buildings described by ardy and Ripley.[15]) A further four buildings listed in these Tables have also to e omitted from the correlation studies since the value of δ/l is known for one or ore walls of these buildings, but not the greatest value of δ/l for the whole structure. hese are buildings [33], [35], [41], and [98]. In addition, the three buildings on ox foundations [26], [34], and [76] have to be omitted owing to the fact that the gidity of such foundations is much greater than that of rafts of normal design. hus there are thirty-seven cases which can be used.

A preliminary examination of these cases shows, as would be expected, that the ngular distortion for a given maximum settlement is smaller for a raft than for a oundation consisting of isolated footings (whether or not the footings are supported n piles). It is also seen that the angular distortion for a given maximum settlement smaller for clays than for sands or fills, owing to the greater variability of these

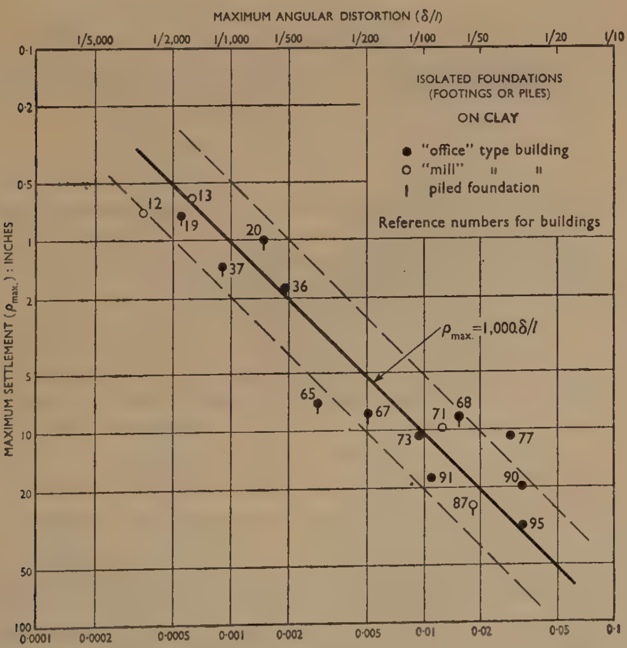


FIG. 4a.—RELATION BETWEEN MAXIMUM ANGULAR DISTORTION AND MAXIMUM SETTLEMENT FOR BUILDINGS ON ISOLATED FOUNDATIONS (FOOTINGS OR PILES) BEARING DIRECTLY ON CLAY

(Excluding cases with deep-seated clay layers)

latter materials. Consequently, in studying the data, a sub-division into the following four classes has been adopted, and the number of cases in each class is also given:—

	Clays	Sands
Isolated foundations	16	11
Raft foundations	7	3

In plotting the data (Figs 4 and 5) a distinction is made between the "office" and the "mill" type of building, but no significant difference appears to exist. A distinction is also made between foundations with and without piles, but here again no very conspicuous differences can be seen. Nor did an examination of the data reveal any dissimilarity between steel and reinforced concrete frame buildings.

For isolated foundations on clay (Fig. 4a) the settlements range from about $\frac{1}{2}$ in. to 30 in. and a reasonably good correlation with angular distortion is found, which may be expressed by the equation:

$$\rho_{\text{max.}} = 1{,}000 \,.\, \delta/l$$

or

$$R = \frac{\delta/l}{\rho_{\text{max.}}} = 1/1{,}000$$

where R is the ratio of distortion to settlement, in the units in.$^{-1}$ (and hence the angular distortion corresponding to a maximum settlement of 1 in.). Similarly, for

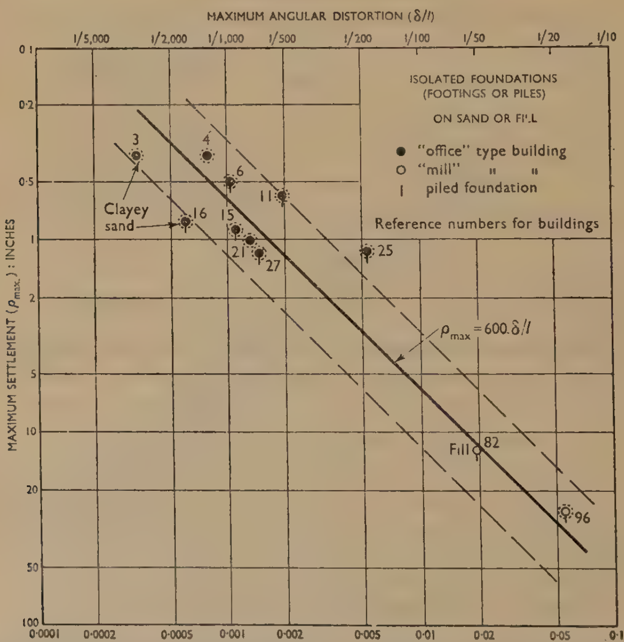


FIG. 4b.—RELATION BETWEEN MAXIMUM ANGULAR DISTORTION AND MAXIMUM SETTLEMENT FOR BUILDINGS ON ISOLATED FOUNDATIONS (FOOTINGS OR PILES) BEARING ON SAND OR FILL

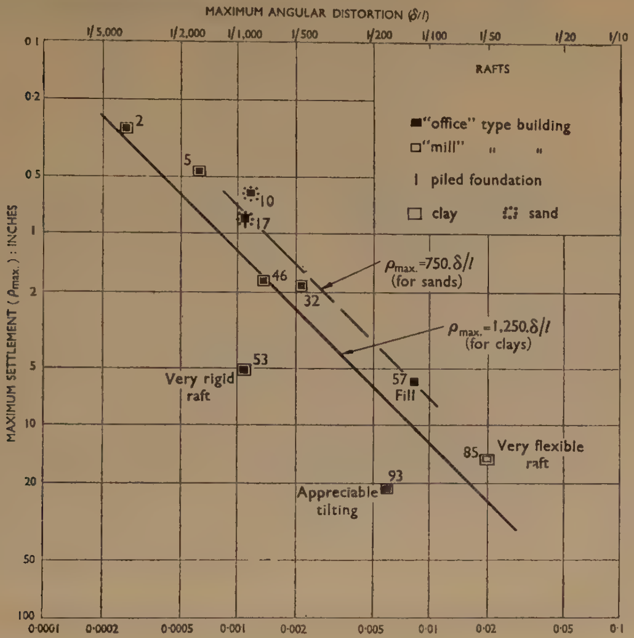


FIG. 5.—RELATION BETWEEN MAXIMUM ANGULAR DISTORTION AND MAXIMUM SETTLEMENT FOR BUILDINGS ON RAFTS

(Excluding cases with deep-seated clay layers)

olated foundations on sand or fill, the points plotted in Fig. 4b indicate a correlation with $R = 1/600$.

The scatter of individual points is naturally quite considerable, but there seems to be little doubt that a statistically significant relation exists between δ/l and $\rho_{\max}$.

The data concerning rafts are plotted in Fig. 5. For clays the seven points are just sufficient to indicate that $R = 1/1{,}250$, although the scatter is large. But for sands, with only two points (and one point for fill) all that can be said is that by analogy with the comparison between sands and clays with isolated foundations, a probable value of R for rafts on sands is:

$$1/R = 1{,}250 \times \frac{600}{1{,}000} = 750$$

One reason for the large scatter in Fig. 5 is evidently that the rigidity of raft foundations can vary considerably. A very stiff raft will undergo less distortion for a given maximum settlement than a very flexible raft. The two most extreme examples of this effect are noted in Fig. 5. Furthermore in building [93] appreciable tilting occurred and the maximum settlement is, as it were, artificially high. Making allowance for these factors the proposed correlation, with $R = 1/1{,}250$, is not unacceptable.

But another factor has been neglected in the foregoing treatment, namely, the width of the foundation. To illustrate this theoretically, consider a flexible circular

foundation on a semi-infinite elastic soil. Boussinesq[16] has shown that in this case the settlements at the centre (maximum) and at the edge (minimum) are:

$$\rho_{\text{max.}} = q \cdot B \cdot \frac{1-\nu^2}{E}$$

$$\rho_{\text{min.}} = \frac{2}{\pi} \cdot q \cdot B \cdot \frac{1-\nu^2}{E}$$

where q is the applied pressure, B the diameter of the circle, ν and E the Poisson's ratio, and Young's modulus of the soil.

Hence, taking the differential settlement between the centre and edge:

$$\delta = q \cdot B \cdot \frac{1-\nu^2}{E}\left(1-\frac{2}{\pi}\right)$$

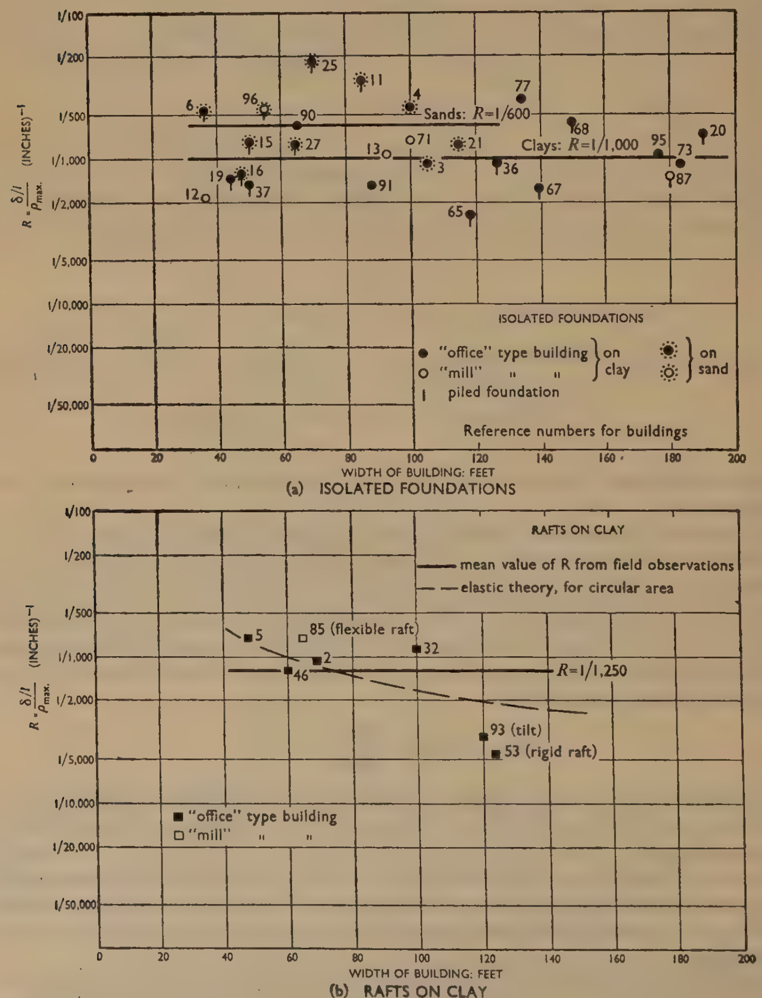


Fig. 6.—Relation between $R = \dfrac{\delta/l}{\rho_{\text{max.}}}$ and width of building

nd, since $l = B/2$, it follows that

$$\delta/l = q \cdot \frac{1 - \nu^2}{E} \cdot 2\left(1 - \frac{2}{\pi}\right)$$

nd, further:

$$\frac{\delta/l}{\rho_{max.}} = \frac{1}{B} \cdot 2\left(1 - \frac{2}{\pi}\right)$$

The above expression is plotted as a broken line in Fig. 6b in which the points epresent raft foundations on clay. The scatter is too great for any definite conlusions to be made, yet it appears that there may be a tendency for the ratio of $/l$ to $\rho_{max.}$ to decrease as the width of the raft increases, approximately in accordance vith elastic theory.* The mean value of this ratio, namely, 1/1,250 must be taken s corresponding to the average width of about 80 ft of the seven rafts included in 'ig. 6b. Fortunately the line representing elastic theory also gives a value of R qual to roughly 1/1,250 for rafts about 80 ft wide. But the variation in R with vidth, as given by elastic theory, must be an extreme case and in practice the effect vould be less marked. A variation of the order of $\pm$ 25% due to the combined ffects of width and rigidity might be considered reasonable (for the range of width f raft normally encountered in building foundations), the lower limit applying to mall rafts of low rigidity and the upper limit to large rafts of high rigidity. For afts on sand the effect of width may be expected to be negligible,[17] although rigidity iust be as important as for clays.

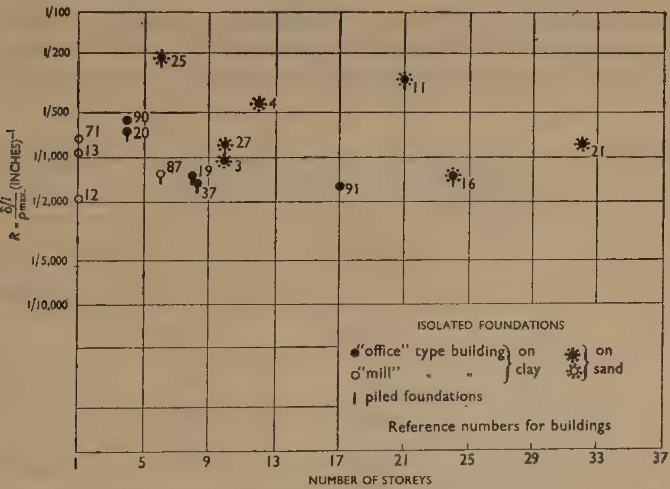


FIG. 7.—RELATION BETWEEN $R = \frac{\delta/l}{\rho_{max.}}$ AND NUMBER OF STOREYS

(BUILDINGS OF UNIFORM HEIGHT, ISOLATED FOUNDATIONS)

* Rectangular foundations on a semi-infinite soil show smaller values of R than for a rcle; whilst if the soil is thin compared with B the ratio is increased. Therefore the use ' a circular area on a semi-infinite soil is convenient and seems to give a reasonable dication of the effect of foundation width. The values of R for buildings [53] and [93] 'e exceptionally low, for reasons already given, and allowance for this fact should be ade in examining Fig. 6b.

For buildings with footings on clay the effect of width will naturally be less pronounced than with rafts since in most buildings the footings cover, at the most, 50% of the plan area. From the available data, which are plotted in Fig. 6a, it will be seen that no trend can be detected. Thus it appears, at least as an approximation, that a value of $R = 1/1{,}000$ can be taken as holding good for buildings on footings on clays, irrespective of the width. Similarly, the effect of width for buildings with footings on sands may be taken as negligible.

A further variable which might be expected to play a significant part in the problem is the rigidity of the superstructure. Where the height of a building is uniform, or substantially so, over its entire plan, the rigidity may be expressed by the number of storeys. Of the buildings in Tables 2 to 5 sixteen with footing foundations have been selected as conforming to this type, and the relevant data are plotted in Fig. 7. There is, perhaps, a tendency for the ratio of δ/l to maximum settlement to decrease as the number of storeys increases; but the evidence is not conclusive. Only five buildings of reasonably uniform height, founded on rafts, are available for a study of this problem, and it is found that the data are insufficient for any trend to be seen. Until more information is obtained it is therefore unwise to accept appreciably larger settlements for multi-storey buildings than for buildings of small height.

The relation between angular distortion and maximum settlement, expressed by the equation:

$$\rho_{\text{max.}} = \frac{1}{R} \cdot \frac{\delta}{l}$$

can now be summarized as in Table 6. Since damage is likely to occur in load-bearing brick walls and in the panels of stanchion and beam frame-buildings when δ/l exceeds 1/300, the corresponding damage limits for $\rho_{\text{max.}}$ can be given for each value of R.

TABLE 6.—VALUES OF R (IN.$^{-1}$) AND CORRESPONDING VALUES OF $\rho_{\text{max.}}$ FOR THE DAMAGE LIMITS, WHEN $\delta/l = 1/300$

		Isolated foundations	Rafts	
			Average	Range
Clays	R	1/1,000	1/1,250	
	$\rho_{\text{max.}}$	3 in.	4 in.	3 in. to 5 in.
Sands	R	1/600	1/750	
	$\rho_{\text{max.}}$	2 in.	$2\frac{1}{2}$ in.	2 in. to 3 in.

It will be shown later that the damage limits of maximum settlements on clays, given in Table 6, are in approximate agreement with results obtained immediately from field observations. But it should be emphasized that the physical basis for settlement damage is the angular distortion; the limiting value of maximum settlement cannot be considered as having the same order of validity.

Greatest differential settlement and angular distortion

The greatest differential settlement Δ is taken as the difference between the maxi-

ıum and minimum settlements of a building, and the correlations between this uantity and the greatest value of δ/l for the building are shown in Figs 8a and 8b. ›f the fifty-one buildings listed in Tables 2 to 5 all may be used in this context, xcept the four where the greatest value of δ/l is not known, namely, buildings [33], 35], [41], and [98]; also building [88] where the settlements are due primarily to a eneral filling. Thus there are forty-six available cases of which thirty-two are on lay and fourteen on sand or fill.

The data for buildings on clay are plotted in Fig. 8a and, apart from three buildings hich have an appreciable component of tilt,* the correlation between Δ and δ/l ıay be expressed by the equation:

$$\Delta = 550 \,.\, \delta/l$$

nd for sands and filling (Fig. 8b):

$$\Delta = 350 \,.\, \delta/l$$

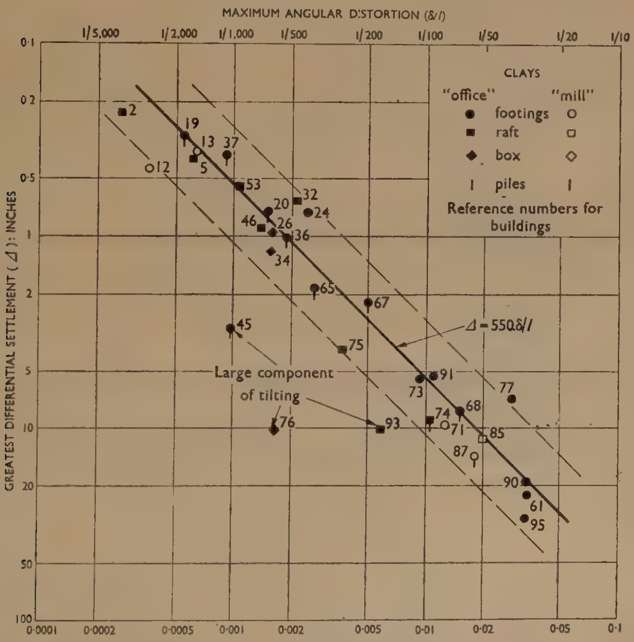


FIG. 8a.—RELATION BETWEEN MAXIMUM ANGULAR DISTORTION AND GREATEST DIFFERENTIAL SETTLEMENT FOR BUILDINGS ON CLAY

Since the angular distortion cannot exceed 1/300 without risk of cracking, it llows that the corresponding damage limits of differential settlement, for all types foundation (but only for the types of building considered in this Paper) are:—

Clays . . . . . . $\Delta = 1\frac{3}{4}$ in.
Sands . . . . . . $\Delta = 1\frac{1}{4}$ in.

ıese results are discussed later, in comparison with the field observations of damage

* Not necessarily *visible* tilting.

TABLE 7.—REFERENCE TABLE, WITH DATA ON SOIL AND FOUNDATION TYPE, MAXIMUM SETTLEMENT AND GREATEST DIFFERENTIAL SETTLEMENT, AND DAMAGE

KEY:—C denotes clay, C_l deep clay layer, S sand, P pile, B box foundation, and T tilt
Figures in brackets refer to damaged buildings.

No.	Building	Soil type	Damage			Maximum settlement: inches						Greatest differential settlement: in.
						Footings			Rafts			
						Load-bearing wall	Frame:		Load-bearing wall	Frame:		
			Functional	Architectural	Structural		"Office"	"Mill"		"Office"	"Mill"	
1	Panoptikon Building, Copenhagen	S					0·15					0·11
2	Apartment Building, Ottawa	C								0·28		0·23
3	Novo Mundo Building, São Paulo	S					0·43					0·32
4	Riscala Building, São Paulo	S					0·45					0·23
5	Mount Sinai Hospital, Toronto	C								0·45		0·40
6	Brick building (5), Vienna	S				0·50P						0·34
7	Apartment building, Vienna	S					0·54					0·53
8	Building XII, Egypt	S					0·57P					0·20
9	Helvetia-Vie Building, Egypt	C								0·59		0·17
10	Thomas Edison Building, São Paulo	S								0·60		0·49
11	Hotel São Paulo	S					0·66P					0·34
12	Fire Testing Station, Elstree	C						0·71				0·45
13	Building III, Egypt	C						0·80				0·80

15	Brick building (20), Vienna	S				0·88P						0·58
16	Azevedo e Villares Bldg, São Paulo	S					0·93P					0·31
17	Banco do Estado, São Paulo	S								0·94P		0·30
18	Texas Exposition Building 43, Dallas	C						0·96				0·34
19	Building XI, Egypt	C					0·96P					0·39
20	Hayden Memorial Library, M.I.T., Boston	C					1·0P					0·75
21	C.B.I. Esplanada, São Paulo	S					1·05					0·43
22	Alumni Pool Building, M.I.T., Boston	C						1·37P				1·0
23	Fair Store, Chicago	C					1·38					
24	Ipiranga Building, São Paulo	C_l					1·38					0·78
25	Building VII, Egypt	S					1·42P					1·10
26	Ohio Bell Telephone Building, Cleveland	C								1·63B		0·94
27	Building X, Egypt	S					1·73P					0·35
28	Maiden Lane Building, New York	S					1·75					
29	Riihimäki Railway Station, Finland	S					1·89P					0·63
30	Bank of China, Hong Kong	C								1·90P		
31	S.I. Claire Fontaine, Lausanne	Fill					1·92					1·43
32	Whitehall Gardens Office, London	C								1·97		0·66
33	Building A	Fill							2·1			1·40

TABLE 7.—*Continued*

No.	Building	Soil type	Damage			Maximum settlement: in.						Greatest differential settlement: in.
						Footings			Rafts			
			Functional	Architectural	Structural	Load-bearing wall	Frame: "Office"	Frame: "Mill"	Load-bearing wall	Frame: "Office"	Frame: "Mill"	
34	New England Mutual Ins. Bldg, Boston	C								2·15B		1·30
35	Building D	C				2·3						0·47
36	Liberty Mutual Ins. Building, Boston	C					2·30P					1·35
37	Building V, Egypt	C					2·36P					0·43
38	Western Bank Note Bldg, Chicago	C					2·50					
39	V.A. Hospital, New Orleans	C_l								2·60P		1·1
40	Central Y.M.C.A. Building, Chicago	C					2·69					
41	Building C	S				2·8						1·03
42	Home Insurance Co. Building, Chicago	C					4·0					0·75
43	Factory Building, Cincinnati	Fill	×	×	×			[3·2]				3·0
44	Judson Building, Chicago	C						3·24				1·44
45	Pacific Gas & Electric Bldg, San Francisco	C_l					3·24P					3·24 T
46	Municipality Bldg, Minieh	C							3·3			1·4
47	Gulf Building, Houston	C								3·4		0·7
48	Apartment Bldg, Geneva	C_l								3·9		

50	Building VIII, Egypt	C		×			[4·3P]					
51	Homoeopathic Hospital, Pittsburgh	C	×	×	×	[4·7]						4·7
52	World's Fair Admin. Bldg, New York	Fill		×			[4·8P]					4·1
53	San Jacinto Monument, Texas	C								5·1		0·6
54	Old Colony Building, Chicago	C	×	×			[5·1]					
55	Latino Americana Bldg, Mexico City	C_1								5·5BP		0·2
56	Building 8, M.I.T., Boston	C					5·7P					
57	Five-storey brick building (4)	Fill		×					[5·8]			5·6
58	World's Fair Communications Bldg, New York	Fill	×	×				[5·8]				4·2
59	Mutual Life Ins. Bldg, Mexico City	C								4·4		
60	Building 6, M.I.T., Boston	C					6·0P					
61	Post Office, Bregenz, Austria	C_1	×	×		[c. 6]						c. 5
62	Ellsmere Place Apartments, New York	C	×	×	×	[6·1P]						6·0
63	Building IX, Egypt	C								6·7		1·3
64	Mixed Law Courts Building, Cairo	—	×	×		[c. 7P]						c. 5
65	Shell Oil Building, San Francisco	C					7·1P					1·9
66	Polk Street Station, Chicago	C	×	×	×	[8·1]						8·0
67	Administration Bldg, M.I.T., Boston	C		×			[8·2P]					2·2
68	Gare Maritime, Le Havre	C	×	×	×		[8·5P]					8·5

TABLE 7.—*Continued*

No.	Building	Soil type	Damage			Maximum settlement: in.						Greatest differential settlement; in.
						Footings			Rafts			
						Load-bearing wall	Frame:		Load-bearing wall	Frame:		
			Functional	Architectural	Structural		"Office"	"Mill"		"Office"	"Mill"	
69	Unity Building, Chicago	C	×	Tilt			[9·1]					9·0 T
70	High Court, Calcutta	C	×	×	×	[9·4]						3·8
71	Warehouse, Kippen, Stirling	C	×	×	×			[9·8]				9·8
72	Great Northern Hotel, Chicago	C	×	×			[10]					
73	Old Board of Trade bldg, Chicago	C	×	×	×	[10]						5·8
74	Charity Hospital, New Orleans	C_1	×	×	×					[10P]		9·0
75	Standard Oil building, San Francisco	C_1								10·1		3·9
76	Nonoalco Power Plant, Mexico City	C									>10·58B	8·5 T
77	Masonic Temple, Chicago	C	×	×	×		[11·8]					7·0
78	Tacoma Building, Chicago	C	×	Tilt			[12]					T
79	Mississippi State Capitol, Jackson, Miss.	C		×	×	[12]						
80	Aircraft Hardware Mfg Co., New York	Fill	×	×				[12]				
81	Filtration Plant, Cleveland, Ohio	Fill	×	×	×					[12]		5·0
82	Factory building, Wembley	Fill	×	×	×			[12·2P]				8·0
83	U.S. Federal Building, Chicago	C	×	×	×				[14]			12

85	Retort House, Weston-super-Mare	C	×		×						[15]	11·5
86	Continental Motors Power Plant, Mich.	Fill	×	Tilt				[15·2P]				12 T
87	U.S. Navy Building No. 76, Philadelphia	C	×	×	×			[16P]				10
88	Locomotive shed, Kerava, Finland	C									16·7	4·5
89	County Building, Chicago	C	×	×	×	[18P]						18
90	National Museum, Ottawa	C	×	×	×	[19·2]						19·2
91	Monadnock Block, Chicago	C	×	×		[21·2]						5·3
92	Factory building, Pacific Coast	C_1	×					[21·5P]				10
93	Apartment building, Chicago	C	×	Tilt						[22·2]		10·3 T
94	Dunwoody Industrial Inst., Minneapolis	—	×	×	×		[24P]					7·0
95	Auditorium, Chicago	C	×	×	×		[27]					26
96	Jurgens Oil Mill, Zwijndrecht	S	×		×			[28P]				28
97	U.S. Custom House, New Orleans	C	×	×	×				[30]			12
98	National Theatre, Mexico City	C	×	×	×					[60]		20

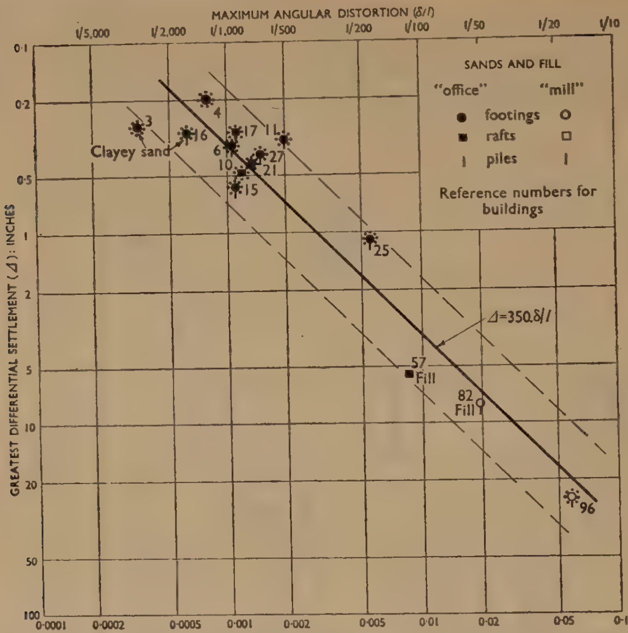


FIG. 8b.—RELATION BETWEEN MAXIMUM ANGULAR DISTORTION AND GREATEST DIFFERENTIAL SETTLEMENT FOR BUILDINGS ON CLAY OR FILLING

in relation to differential settlements. There seems to be little justification, from the data, in making a distinction between rafts and isolated foundations; presumably a raft reduces by roughly equal amounts both the angular distortion and the differential settlement (as compared with the same building on footings). Even the two box foundations [26] and [34] lie practically on the line representing the average correlation for all types of foundation, as will be seen in Fig. 8a.

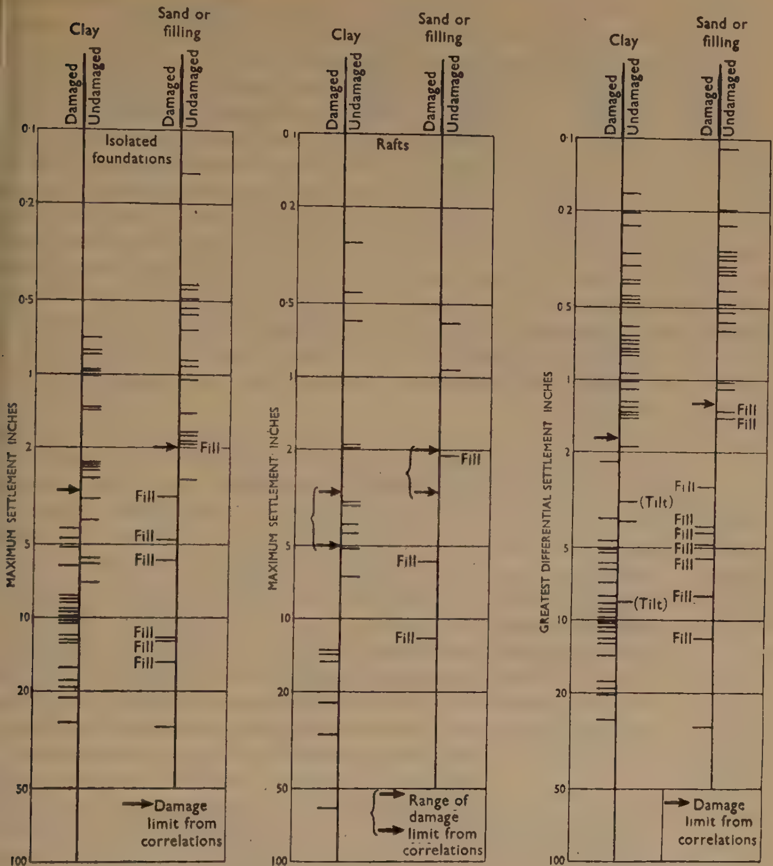
It is worth noting that the values of Δ given above imply that the points of maximum and minimum settlements in a building are, on the average, not closer than 45 ft with clays or 30 ft with sands; and that for foundations on clay the differential settlement between adjacent columns (typically spaced about 20 ft apart) is about one-third to one-half of the greatest differential settlement in the whole building whereas for sands this ratio is likely to be about two-thirds.

Similarly, it may be useful to note that the greatest differential settlements in buildings with isolated foundations average about 60% of the maximum settlements whilst with raft foundations they vary from roughly 35 to 60% of the maximum. In this respect, there seem to be no very marked differences between sands and clays.

DATA ON MAXIMUM SETTLEMENT

Information on the maximum settlement and the presence or absence of damage in ninety-eight buildings is set out in Table 7; data for eighty-three cases, where the soil conditions are known and where the building neither has a box foundation nor is situated above a deep-seated clay layer,* are plotted in Fig. 9a.

* Building [88] is also omitted for reasons given above.



(a) RELATED TO MAXIMUM SETTLEMENT (b) RELATED TO DIFFERENTIAL SETTLEMENT

FIG. 9.—FIELD EVIDENCE ON DAMAGE

he dates of construction of the ninety-eight buildings range from 1860 to 1952 the incidence of damage may be summarized as follows:—

	Date of completion of building				Total
	1860–99	1900–29	1930–39	1940–52	
amaged . .	4	17	17	20	58
aged . .	16	15	8	1	40

change in distribution, between damaged and undamaged buildings, in the four ods is due partly to improved foundation design but more to the fact that many

of the settlement observations in the past 20 years have been made irrespective of whether or not the buildings were known or suspected to have large settlements.

On examining Fig. 9a perhaps the most striking fact is that, with two exceptions, all the eighteen buildings on sand have settled less than 2 in. Building [41] has settled 2¾ in. but it also is undamaged, whilst the only case of damage on sands is building [96] where the ultimate bearing capacity of the piles was probably exceeded and the settlements amounted to more than 2 ft. With virtually no cases of damage on sands it is clearly impossible to derive any definite conclusions regarding the upper limit of allowable maximum settlement. Yet it is to be noted that the value of 2 in. for footings on sand, obtained from the correlation ratio R and the limiting angular distortion 1/300, is at least confirmed to the extent that no cases of damage are known with this or smaller settlements. There are several examples of damage in buildings with footings on fill where the settlements exceed 3 in. Nothing can be said regarding the damage limit of 3 in. for rafts on sand, obtained from R and δ/l; but for rafts on fills, the field evidence suggests that the limiting settlement is likely to be between 2 and 6 in.

There is considerably more information on clays—of the thirty-nine buildings with footings on clay, twenty-one have been damaged. It will be recalled that from the correlation ratio the limiting maximum settlement, for this type of foundation, was found to be 3 in. From Table 7 and Fig. 9a it will be seen that no less than six buildings have settlements between 2½ and 3½ in. and none have suffered any damage; whilst there are three buildings with settlements between 4½ and 5½ in., two of which have cracked and one has also suffered structural damage.

It therefore seems to be possible to conclude that the limits of 3 in. is acceptable, even if it is perhaps slightly conservative. But there are four buildings with settlements of 4 to 7 in. which appear to contradict this statement. In two of these, [56] and [60], the published data are very meagre; the differential settlements are not known and the presumption that no damage has occurred is based only on the fact that no statement concerning the existence of damage has been made. As for building [65], the greatest differential settlement and the maximum angular distortion (1/370) are compatible with the absence of cracking, and this is an example of the way in which a statistical approach breaks down in occasional particular instances. However, against these undamaged (or presumed undamaged) buildings there are two cases, including one [61] on a deep-seated clay layer, where damage has occurred with settlements of 6 to 7 in. All the buildings with footings on clay which have settled more than 7 in. have been cracked and the majority have had structural damage as well.

For buildings with raft foundations on clay the limiting settlement predicted from the correlation ratio ranged between about 3 and 5 in. depending on the rigidity and size of the raft. The evidence in Table 7 and Fig. 9a suggests that these values are conservative, since there are five buildings with settlements in this range none of which have suffered damage. Unfortunately, however, the conservative nature of the predicted values cannot be examined quantitatively from the field data, since the next cases, in order of increasing settlement, have maximum settlements of 14 in. and have been badly damaged, as might be expected.

Summing up the data on maximum settlement, it may be said that the limits derived from the correlation ratios and $\delta/l = 1/300$ are reasonable, although probably rather conservative. It should again be emphasized that the use of maximum settlement as a criterion can be justified only as a guide in the design of foundations bearing directly on sand or clay; it cannot apply where the settlements are due

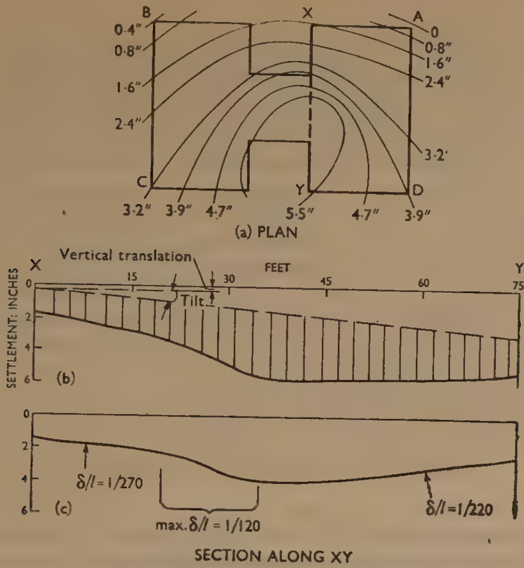


FIG. 10.—SETTLEMENTS OF BUILDING [57]

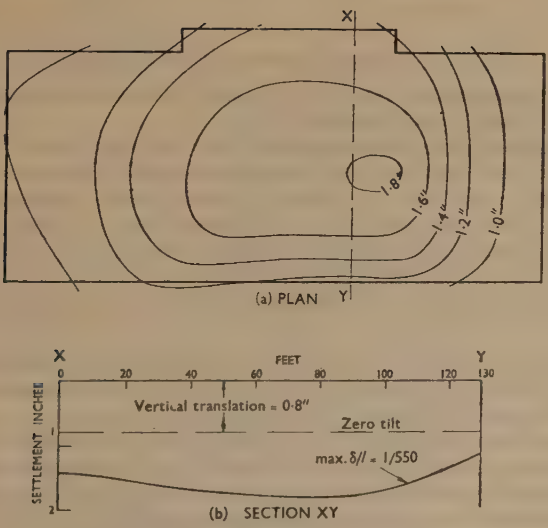


FIG. 11.—SETTLEMENTS OF BUILDING [36]

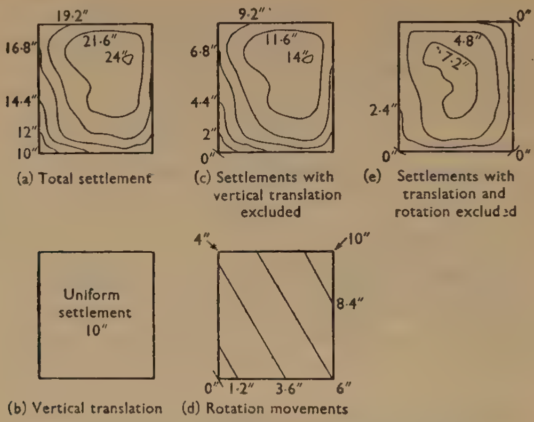


FIG. 12.—SETTLEMENTS OF BUILDING [87]

essentially to the consolidation of deep-seated clay layers. Also, as an overall consideration, the foregoing criteria of maximum settlement are relevant only for normal foundation conditions and for the types of building considered in this Paper. There are individual cases in which these limiting settlements will not be even approximately valid as, for example, in building [88] where large but fairly uniform settlements are caused by a fill covering a wide area. In contrast, the limiting settlement might obviously be smaller than the foregoing values if a building was erected alongside an existing structure and bonded with it, or if an architectural feature was aligned with a corresponding feature in the older building.

DATA ON GREATEST DIFFERENTIAL SETTLEMENT

Information concerning the greatest differential settlement of seventy-nine buildings is set out in Table 7 and plotted in Fig. 9b.

For buildings on clay the field data suggest that the limiting value of Δ is about 2 in. since none of the buildings with smaller differential settlements have suffered damage, and all except three of the buildings with greater values of Δ have been cracked. But of these [45] and [76] can be disregarded since their high differential settlements have resulted largely from tilting, and the third exception, namely building [75], may possibly be explained by the very slow rate of settlement, the differential settlement of 3·9 in. not being attained until about 30 years after construction.

For buildings on sand or fill the limiting value of Δ seems, from the evidence in Fig. 9b, to lie between $1\frac{1}{2}$ and 3 in.; there being no cases of damage when Δ is less than $1\frac{1}{2}$ in. and no cases without damage when Δ exceeds 3 in.

From the correlations between greatest differential settlement and maximum angular distortion, and using the value of 1/300 for the limiting distortion, damage limits of Δ equal to $1\frac{3}{4}$ and $1\frac{1}{4}$ in. are obtained for buildings on clay and sand respectively. These values are shown in Fig. 9b and they are in moderately close agreement with the field evidence, erring a little on the side of safety.

Summary of damage limits

Cracking of the panels in frame buildings of the traditional type, or of the walls in ad-bearing wall buildings, is likely to occur if the angular distortion exceeds 1/300. ith a typical span of 20 ft this distortion corresponds to a differential movement $\frac{3}{4}$ in. Structural damage to the stanchions and beams is likely to occur if the igular distortion exceeds 1/150.

From an examination of a number of settlement records it is found that the greatest fferential settlement in a building, associated with a maximum angular distortion 1/300, is typically about $1\frac{3}{4}$ in. if the foundation is on clay and about $1\frac{1}{4}$ in. if the undation is on sand or filling.

For a given angular distortion, or greatest differential settlement, the maximum ttlement of a building is greater with a raft foundation than with footings. Corre- onding to the angular distortion of 1/300 (or the greatest differential settlement $1\frac{3}{4}$ in.) the damage limits of maximum settlement for buildings on clays are pically about 3 in. for footings and 4 in. for rafts. Similarly, the damage limits maximum settlements for buildings on sands are about 2 in. for footings and in. for rafts.

Owing to the considerable variation in the rigidity of different rafts, and owing the effect of width, the damage limits of maximum settlements for buildings on fts will be subject to appreciable variations about the average figure given above. iggested ranges of values are given in Table 8 where, for convenience, the con- isions regarding damage limits are summarized.

Table 8.—Damage limits for load-bearing walls or for the panels in traditional-type frame buildings

Criterion		Isolated foundations	Rafts
Angular distortion		1/300	
reatest differential settlement:	Clays	$1\frac{3}{4}$ in.	
	Sands	$1\frac{1}{4}$ in.	
Maximum settlement:	Clays	3 in.	3 to 5 in.
	Sands	2 in.	2 to 3 in.

The criteria in Table 8 are subject to the following principal limitations:—

General.—The criteria have been derived from an observational study of buildings of load-bearing wall construction, and steel or reinforced concrete frame buildings with conventional panel walls, but without diagonal bracing. They are intended as no more than a guide for day-to-day work in designing typical foundations for such buildings, and in certain cases they may be over-ruled by visual or other considerations.

Angular distortions.—Where the settlements increase very slowly with time, cracking may not commence until somewhat larger angular distortions have been developed in the building.

Greatest differential settlement.—The values are probably slightly conservative.

Maximum settlement.—The values do not apply to buildings where the settlements arise essentially from the consolidation of deep-seated clay layers, nor to a building which is erected immediately adjacent to an existing structure to which it has to be bonded or aligned, nor where the settlement will be due largely to a general filling and not to the building itself.

Factors of safety

Although tending to err on the side of safety, the limits given in Table 8 should not be exceeded in design without special investigations or knowledge from previous local experience. In most cases the engineer will apply a factor of safety before arriving at appropriate design limits. For angular distortion, in view of the uncertainties in calculating settlement contours, a factor of not less than 1·5 is suggested; this gives a limit of about 1/500, and where it is particularly desired to avoid any settlement damage the limit might well be decreased to 1/1,000.

For greatest differential settlements a factor of safety of 1·25 may be considered sufficient; this gives design limits of 1½ in. and 1 in. for foundations on clay and sand respectively.

For maximum settlements the same factor of safety would result in the following design limits:—

Isolated foundations on clay	2½ in.
Isolated foundations on sand	1½ in.
Rafts on clay	2½ to 4 in.
Rafts on sand	1½ to 2½ in.

For box foundations on clay the allowable maximum settlements can exceed 4 in. provided, of course, that such settlements will not give rise to a visible tilt in the building or other undesirable consequences.

The foregoing suggestions for values of the design limits are all subject to the same limitations as are the damage limits.

Reference to Appendix II show that these design limits are in broad agreement with those which have been put forward from time to time by experienced foundation engineers. Yet much remains to be discovered, particularly in relation to the characteristics of curtain walling, and to the applicability of laboratory tests in assessing the limiting distortions of various types of panels and partitions. Above all, further settlement observations and performance records of buildings are required. But a good deal of published information undoubtedly exists of which the Authors are not aware, and it is probable that a still more detailed examination of the case records used in the present Paper would yield additional data of value.

It should finally be emphasized that the design limits given above are in no way to be regarded as a set of rigid rules. It would appear to be unlikely that any settlement damage to a building will occur so long as these limits are not exceeded. But in individual cases the engineer will use his judgement and experience which, under certain conditions, may lead him to adopt very different criteria. Nevertheless it is hoped that, by bringing together and making a preliminary analysis of this collection of field data, a contribution may have been made towards the solution of the problem of deciding what, for any given building, may properly be considered as the allowable settlement.

Acknowledgements

Much of the data upon which the Paper is based were collected by Dr MacDonald, while a research student in the Department of Civil Engineering, Imperial College, University of London, and are contained in a thesis.[18] This work was made possible by successive awards of an Athlone Fellowship and a Special Scholarship of the National Research Council of Canada. Some of the information has not been previously published, and for access to these data the Authors wish to acknowledge the Director of Building Research, Department of Scientific and Industrial Research, Dean R. M. Hardy, Professors K. Terzaghi, R. B. Peck, H. Lundgren, K. V. Helenelund, M. Vargas, and L. Zeevaert, and the Director of the Division of Building Research, National Research Council of Canada.

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APPENDIX I

Details of three case records

Five-storey brick building [57]

This five-storey brick load-bearing wall building was founded on a 4-ft concrete raft. The settlement contours are shown in Fig. 10a and it is known that cracking occurred in the walls on section XY, the settlement profile of which is given in Fig. 10b. To correct for the bodily tilt and vertical translation of the building, first imagine that the foundation is lowered uniformly by the settlement at corner A, namely, 0·2 in. Then imagine that the foundation is rotated about AD in such a manner that corners B and C both fall to a displacement of 0·4 in. Thirdly, imagine that the plane is finally rotated about AB to increase the settlement of corner C from 0·4 to 3·2 in. This operation will cause D to reach a total settlement of 3·0 in. These figures are assumed to represent the displacements of the foundation due to rigid-body rotation and vertical displacement. The corresponding displacements at X and Y are, by simple interpolation, 0·3 and 3·1 in. respectively. The base line joining these displacements is drawn in Fig. 10b, and the settlements with the rigid body rotation eliminated are those shown by the shaded area and plotted in Fig. 10c.

From this corrected settlement profile the maximum value of δ/l, measured over a reasonable length such as 15 ft, is found to be 1/120. But the walls were definitely cracked all along their length and therefore it will be seen from Fig. 10c that an angular distortion of 1/270 was sufficient to cause damage.

The data for this building may thus be summarized as follows:—

Maximum settlement = 5·8 in.
Greatest differential = 5·6 in.
Maximum δ/l = 1/120
δ/l causing cracking = 1/270

All these figures apply at the same time.

Liberty Mutual Insurance Company Building, Boston [36]

This large frame building, with a maximum height of ten storeys, is founded on combined footings supported on Gow caissons. There is a basement of variable depth over the plan of the building, and the average net pressure is small. The caissons (or piers) extend to the stiff crust of the Boston Clay, at a depth of 40 ft below ground level, and beneath the crust is a bed of soft clay 60 ft thick. Complete settlement contours (Fig. 11a) have been published for a time 4½ years after the beginning of construction, when the maximum settlement was 1·85 in. and the greatest differential was 1·02 in. A settlement profile on section XY is plotted in Fig. 11b and since the tilting of the building is almost zero, there is no need to correct this curve. The maximum value of δ/l is 1/550 and no

mage was reported. The foregoing values of δ/l, maximum settlement, and greatest fferential settlement are those given in Table 4. However, settlement observations ıve been continued subsequently, but no further settlement contours have been pubhed. Therefore no further values of δ/l can be evaluated, but the maximum settlement ıd greatest differential settlement are known 10 years after construction, and the values e 2·30 and 1·35 in. respectively. Since no damage is reported for this later time of years after construction, it is these larger values that are entered in Table 7.

nited States Navy Building No. 76, *Philadelphia* [87]

This building is a six-storey reinforced concrete flat-slab warehouse structure, 0 ft × 180 ft in plan, with a piled foundation in silt. About 4 months after construcon some of the wall panels and all the floors were observed to be cracked due to the ttlements, the maximum value of which, at this time, was 16 in.; the greatest differential ttlement being 10 in. These figures have been given in Table 7. No settlement conurs were published for this stage of the settlement, but 3 years later, just before the ilding was underpinned, a complete set of contours was obtained and published. These ntours are shown in Fig. 12a. The vertical translation is 10 in. (Fig. 12b) and the ttlement pattern with this component excluded is given in Fig. 12c. The components rotation are given in Fig. 12d and the settlement pattern with both translation and tation excluded is plotted in Fig. 12e. From this final pattern of contours the maximum lue of δ/l is found to be 1/55. At this time the maximum settlement and greatest fferential were 24 and 14 in. respectively, and these values are given in Table 4.

APPENDIX II

Previous statements concerning allowable settlements of buildings

The Authors have not made an exhaustive search for statements concerning the allowle settlements of buildings, but it is thought that the following summary includes most the experienced opinions that have been published.

Baumann [19] (1873), referring to load-bearing wall buildings, suggested 1½ in. as the dvisable settlement".

Jenney [20, 21] (1885 and 1891). In the earlier of these two Papers Jenney says, in conxion with the Home Insurance Building in Chicago [42], that a frame structure with ›tings on the compressible clay of Chicago "must necessarily settle, the problem being reduce the settlement to a moderate amount, say, from 2 to 3 inches, and to make the tlement practically uniform." In the later Paper, still referring to frame buildings on ›tings in Chicago, he says "experience teaches that a load of 3,000 lb./sq. ft. will cause ompression of about 2 inches, and that it is desirable not materially to increase this load the compressions may not be sufficiently uniform."

It should be pointed out, however, that Chicago engineers of this period were prepared, ough force of circumstances, to accept considerably larger settlements and a certain ount of trouble in consequence. For example Corydon Purdy [22] implies that settlents of 3 to 5 in. were quite usual, and Jenney himself allowed for a settlement of 4½ in. the Fair Store Building of 1892.

Simpson [23] (1934), referring to frame buildings and probably implying raft foundations clay, mentioned that 2 in. was an allowable differential settlement and that the maxim settlement could be 4 to 5 in.

Terzaghi [24] (1935), stated as a result of his investigations on brick load-bearing walls, which reference has been made in the present Paper, that the greatest angular distion which they can withstand is approximately 0·0035 (1/285). In general, he states t an average settlement of 1 in. must be considered normal, but that reinforced conte frame buildings may suffer some damage if the settlement exceeds 2 in.

Terzaghi and Peck [25] (1948) state that "most ordinary structures, such as office builds, apartment houses, or factories, can withstand a differential settlement between acent columns of ¾ inch." It may be noted that, with the usual bay spans of about ft, this differential settlement corresponds to an angular distortion of 1/320. Terzaghi Peck also state that the differential settlement for buildings with footings on sand is ikely to exceed 75% of the maximum settlement, and therefore indicate that a maxim settlement of 1 in. should be used in design of such buildings. For raft foundations sand they point out that the ratio of differential to maximum settlement is not more n one-half the corresponding ratio for footings, and therefore the allowable maximum

settlement for buildings on rafts underlain by sand is 2 in. Finally these Authors state that, whereas the differential settlement between the adjacent columns of a building on a raft on sand can almost equal the greatest differential settlement, yet with rafts on clay the difference between two adjacent columns never exceeds a small fraction of the greatest differential settlement. "Therefore the tolerable differential settlement for rafts on clay is very much greater than that for rafts on sand."

Tschebotarioff [26] (1951), after pointing out that the amount of deflexion that buildings can undergo can be found only by observations of full-scale structures in the field, states "So long as the total settlement does not exceed 2 or 3 inches, no damage is to be expected with most buildings. The differential settlement then does not exceed an inch or so and it would appear that the masonry superstructure of most buildings can safely deflect by the necessary amount."

Ward and Green (1952),[27] from field observations on brick houses and from tests on panel walls made at the Building Research Station, conclude that a relative settlement of ¼ in. in 10 ft is not excessive. This is equal to an angular distortion of 1/480.

Meyerhof[12] (1953), from considerations of the same tests, suggests an angular distortion of 1/300 as the permissible value for open frames, and 1/1,000 for frames with wall panels. For load-bearing brick walls or buildings with continuous brick cladding he suggests a central deflexion of 1/2,000 of the span, corresponding to an angular distortion of about 1/600.

APPENDIX III

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The Paper, which was received on 22 February, 1956, is accompanied by one photograph and fifteen sheets of diagrams, from which the Figures in the text have been prepared, and by three Appendices.

Discussion

Mr E. O. Measor (Partner, Scott & Wilson, Kirkpatrick & Partners, Consulting Engineers) observed that the Paper was the first comprehensive treatment of the subject.

The development during the past 25 years of methods of calculation of settlement had made a profound difference to foundation engineering, but the effect had hitherto enabled civil engineers to avoid dangers rather than to make more economic designs. The object of engineering science should surely be to enable engineers to take greater and not less calculated risks. The calculations of settlement had unfortunately often served rather to frighten them; recent foundation designs had often been more expensive than previously when they could proceed rather more lightheartedly.

On pp. 728 and 729 the Authors had referred to calculations of stresses in building frames and had eventually dismissed such calculations in their final paragraph as of little use. That was correct when applied to elastic calculations. In considering stresses arising from settlement, ultimate strength calculations based on plastic theory were realistic. That was illustrated by a problem with which his firm dealt a few years previously. A single-storey warehouse had been built on recent filling adjacent to Malta Harbour; both actual and future predictable settlements were large. The elastic calculations applied

e single-storey frame indicated that it had been stressed far beyond safe limits. Never-eless, plastic calculations had shown that its strength was unaffected. Cracking of ternal walls and adjustments to windows had been inevitable, but for such a building at had had to be accepted. That led him to a point he wished to make.

The Authors had divided buildings into two types—mill, and office buildings—but ving divided them, had said subsequently that there was no significant difference be-en them! However, buildings should be classified according to their usage in relation the importance of wall cracking. In hospitals, which were an extreme case, cracking of nel walls was unhygienic and unacceptable. In flats, hotels, and office blocks, wall acking was not welcome, but tended to occur not because of failures by the engineer or e foundations, but because of shrinkages. Client co-operation and education were very portant, for usually in the design and construction stage great pressure was put on the gineer to cut his designs to the minimum to reduce cost; however, when the building s completed he was summoned to inspect minute cracks! If industrial buildings were be made economic, cracking of facing walls and partitions must sometimes be risked, ce the difference in cost between foundations to guarantee no cracking and those that ght lead to cracking could be considerable.

In judging those matters, the criteria which the Authors had reached from their re-rches were very valuable, and in the course of time much might be added to the undness of the evidence on which they were founded. The criteria were, however, sed principally upon walls, but frames could not be ignored either. Criteria for the mes were vital because they were more important; with the development of curtain lling and light-weight non-rigid partitions greater settlements might be acceptable for e partitions and external walls than at present, but not for the frames.

The Authors had given a relation between total differential settlement and angular tortion, and had concluded that total settlement causing failure was relatively inde-ndent of the size of building. That was probably not surprising for architectural mage, because there were two elements—the possibility of local irreularity between acent columns, and the overall curvature of the building—and the panel wall was sitive to both types, and the two added up. In the case of the structure, with a large lding a much greater overall settlement than 3 in. could be obtained, without affecting structural strength. Criteria for permissible structural settlements were needed since se ultimately were the engineer's greatest responsibility.

With regard to floors, which could be quite troublesome he had recently dealt with the blem of a ground-bearing floor subject to the effect of the settlement of the column ndations; he had to decide how much the columns could be allowed to settle, particu-y with a floor on which there would be heavy trucking. That was a separate problem, it was important in industrial buildings.

Dr L. F. Cooling (Head of Soil Mechanics Division, Building Research Station) said t is was no mean feat to have collected together such complete records for as many as ety-eight buildings. It was even more of an achievement to have analysed and ested the results so thoroughly as to be able to specify damage limits and to relate them such factors as angular distortion and differential settlement. It was of particular rest that the Authors had found that damage occurred if the angular distortion ex-ded 1/300, a value which agreed with tests carried out in the laboratory of the Building earch Station on the distortion of framed panel walls.

Although perhaps a little outside the scope of the Paper, he wondered if the Authors, he course of their examination of the ninety-eight examples, had found sufficient rmation on the soil side to permit any comparison between performance of the building the probable aim of the designer. In so far as those buildings had been designed at it seemed likely that the approach would have been based on permissible bearing sure, and he wondered if it was possible to give any broad indication of the type of s where the bearing-capacity approach had failed to work. For instance, a large portion of the damaged buildings seemed to have been associated with clays; was it

possible to say whether the damaging settlements had been due to plastic yield of the ground through using a factor of safety on bearing capacity rather lower than ought to have been chosen for that particular type of soil? Or were the cases of damage more closely linked with the trend to construct bigger and bigger buildings, and the lack of appreciation of the fact that the larger buildings would lead to larger settlements even if the bearing pressure was kept below the accepted value?

In the final section headed "Factors of Safety" the Authors gave some suggestions on foundation design but it was not clear to him how it was intended that those should be used. It was obviously not practicable to base a design approach on permissible angular distortion and he wondered if the following steps outlined the procedure which the Authors had in mind.

Having obtained test results on samples of soil and using data on the size, shape, and loading, etc., of the structure, the first step was to make a theoretical estimate of the maximum total settlement assuming that the structure was completely flexible. If that value was less than 2½ in. then the design could be considered satisfactory. If the maximum total settlement exceeded 2½ in., and that value could not be decreased by spreading the load, then much more consideration was needed; it became necessary to look into the theoretical pattern of differential settlement and then to see how stiffness and rigidity had to be introduced into the structure to keep down the distortion.

Would Professor Skempton agree that if a theoretical analysis, carried out on the assumption of a flexible loaded area, gave an *average* total settlement (a value which could be taken as the settlement of a rigid foundation) exceeding 4 or 5 in. it would be necessary either to reduce the net load by partial flotation, to use piles, or perhaps to move to another site?

Mr R. W. Souza (Research Student, Imperial College of Science and Technology) said that during the past few months at Imperial College he had been examining data to discover if there was any relationship between settlements causing damage and the rate of settlement. On sands settlement was usually completed at the end of construction or soon after. Therefore, he limited his remarks to structures on clay, and since most of the critical data applied to footings, the data mainly concerned buildings with footings on clays.

There were no reported cases of damage for buildings with a maximum settlement of 3 in. or less; equally important, there were no undamaged buildings on footings with a maximum settlement of 8 in. or more. Between those two limits, there were fifteen cases, nine damaged and six with no reported damage. Of the damaged buildings trouble had occurred in most (43, 51, 54, 58, 61, and 62) during construction or immediately after, and in the remaining three (50, 52, and 64) damage was reported within [illegible] years of the start of construction. However, in most of the six undamaged buildings with settlements of more than 3 in. (which represented exceptions to the Authors' suggested limit for isolated foundations on clay) settlements had been much slower. The Home Insurance Building (42) had taken 4 years to reach a maximum settlement of 4 in. and the remaining five buildings in the group (44, 45, 56, 60, and 65) had required at least 12 years to reach their reported settlements. In addition, the Monadnock Building in Chicago (91), which the Authors listed as damaged when its settlement was 21 in., was reported free of any visible cracks at a settlement of 5 in. which occurred in 2½ years. Those data suggested that time or rate of settlement was significant in determining the "allowable" settlement. The Authors' damage limit of 3 in. for isolated foundations on clay was applicable at the end of construction, and larger settlements might not cause damage if they occurred during a period of 2 or more years.

Unfortunately, there were insufficient cases to enable the reasons for that effect to be studied in detail. However, as the Authors had pointed out, slow rates of strain might lead to relief of stress in the building materials. In addition, there was a possibility, for which some evidence existed, that the ratio of angular distortion to maximum settlement

nded to be lower if the rate of settlement was very slow. That might result from such w rates of settlement being associated with thick clay deposits.

The data on those points, in relation to raft foundations, were too scanty for any finite conclusion to be made, but there was some evidence that there should be no preciable difference in that respect between rafts and isolated footings. It might be ggested that with rafts also the limits for maximum settlements could be increased if e rate of settlement was slow.

Of the buildings within the range of maximum settlement of between 3 and 12 in., ere were only five that had been underpinned, four being underpinned during con-uction and the fifth before occupation of the building. After 2 years there was only one se of underpinning and the settlement of the building at that time had been at least in. Although there were some outstanding exceptions most of the buildings with a ximum settlement of more than 12 in. had either been underpinned, condemned as safe, or partially demolished.

There were a few corrections to be made to the data given in the Paper.* The Home surance Building (42) which the Authors had listed at a maximum of 3 in. had been ported by strong implication to have reached a maximum settlement of 4 to 5 in. (23a.) e source reference of the Factory Building, Cincinnati (43) contained the implication at the portion of the building on gravel had not settled appreciably; therefore the ferential settlement was 3 in. instead of the 2 in. listed by the Authors. The Mutual e Insurance Building (59), Mexico City had had a settlement of 4·4 in. relative to a near ilding and 5·9 relative to a distant point. Since they were interested in settlement th respect to adjacent buildings and ground levels, he felt that 4·4 in. would be more propriate.

One additional case might be mentioned. The Dade County Court House in Miami, rida, a three- and six-storey building with a twenty-eight-storey tower, founded on ead footings over coral rock and sand, had reached a 4·5 in. maximum and a 3 in. ferential settlement during construction and the decision to underpin the structure l been made. The settlements had increased to 6 in. by the time underpinning was gun. The data for that case could be found in a recent work.[99]

Dr H. Q. Golder (Director, Soil Mechanics Limited) observed that Professor Skemp- had rather decried what he had described as the tentative nature of the Paper, but d that he agreed with Dr Cooling that it was the completeness of the Paper which made cussion difficult. There was indeed little left for one to say. However, the Paper l one very noticeable virtue which he thought was probably not realized by the Authors, l he illustrated that by relating a story which showed that many engineers did not ieve that buildings settled at all—particularly mechanical engineers. If one talked them about settlement, they said, "Oh, no; no settlement." If pressed for a figure, y would talk about three or four thousandths of an inch, which meant nothing to a ndation engineer. After the Paper, in which there were ninety-eight carefully docu-nted cases of settlement, he thought that civil engineers were going to accept the fact t buildings did settle, and that it would then be much easier to talk to them about the ount of settlement which was allowable.

He thought that the point which needed to be made was that settlement was the normal dition. There was nothing abnormal about settlement. All buildings settled; the portant question was how much was the settlement going to be?

t was interesting that the Paper did not discuss causes of settlement, but only the effect. m that point of view the figures given could be useful in, say, underpinning work. e naturally did not allow settlement in underpinning if at all possible, but sometimes a

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These corrections have since been made.—SEC.

[99] E. A. Prentis and L. White, "Underpinning." 2nd edn, Colombia Univ., New k, 1950.

cheaper method might be used if there was some idea of what settlements would be allowable.

His final point was that the allowable settlement was sometimes decided not by the structural damage but by the use to which the building was put. That had also been mentioned by Professor Skempton, who had called it functional damage. In Holland it had been necessary to underpin a building which contained all the telephone instruments for The Hague. The building was on sand and it was necessary to make an excavation next-door to it, so chemical underpinning had been used. It had been pointed out that a settlement of 4 mm would throw all the telephones out of action. The work had been carried out without any trouble but it had been some consolation to know that even if anything had gone wrong, nobody could ring him up and ask him what to do next.

Mr G. M. J. Williams (Scott and Wilson, Kirkpatrick and Partners) said that Dr Skempton had already stressed that his simple criteria, valuable as they were, should be used with caution, because exceptions could occur quite frequently; a very common exception arose in framed buildings with upper structures supported by columns which rested on relatively small foundations, whilst the ground floor consisted of a ground-bearing slab supporting the ground-floor partitions. In such cases almost any settlement of the column foundations would produce differential settlement between the columns and the ground-floor partitions; cracks could occur at values of δ/l far less than 1/300. There was an added danger, of course, that the load from the first floor was thrown on to the top partitions.

A good example occurred in the building which Mr Measor had mentioned. That building had been framed in reinforced concrete with columns supported on pad footings which rested on a layer of loose material which in turn overlay a layer of soft material; the bearing pressure had been about 2 tons/sq. ft. There had been about 4 in. differential settlement from the centre of the floors to the corners of the building, but the value of δ/l between individual columns did not seem to have exceeded about 1/220.

Fig. 3 suggested that the panels would crack and that was certainly the case. There had been cracks all over the building, into some of which one could put one's hand. In one place also there had been quite a serious structural failure not normally expected for such a low value of δ/l.

The first floor was of beam-and-slab construction, the beams being carried on columns which rested on quite small foundations. The ground-floor slab was a simple ground-bearing slab with the lightest conceivable steel mesh in it, and at one point there was a masonry partition built on the ground floor, and tightly pinned up to the slab above, one or two beams passing through it. Although the value of δ/l for the panel containing that partition was quite small, there had been a considerable differential settlement between the column footings and the ground floor. That had transferred loads from the first-floor slab to the top of the partition and the upper surface of the slab had cracked where it passed over the partition. There had been a crack in the floor above which was up to ½ in. wide, and it had gone across at least one beam.

That building had also taught another lesson. It had been decided that about two-thirds of the settlement was complete and that the remainder was going to come quite slowly, and consequently there was no necessity to underpin it. The clients had consequently been advised to modify the partitions so that they were no longer pinned up to the first floor, to fill in the cracks in the walls and make good the other damage, and to go on using the building and keep a careful eye on it. That had been 5 years ago, and no further serious trouble had been reported.

The moral was that if that building had been designed originally by an expert he would probably have put it on piles, and the cost would have been increased by about £15,000. The cost of modifying the partitions and repairing the cracks could not have exceeded £1,000, so that the owners had got a very economical building.

He thought that might be borne in mind, because often when it was certain that any damage from settlement would be largely confined to the non-load-bearing parts of

ructure and that serious frame-damage was unlikely, it was much cheaper to provide nple foundations and to allow for repairing the effects of settlement as they arose. He d found that particularly applicable to schools, which always had to be done cheaply, d where redecoration every few years was expected, so that there was an opportunity r making good cracks and minor damage, and where it was known they were unoccupied r quite long periods at regular intervals, so that work could be done without inconnience.

Dr Skempton had only just mentioned the effects of time in the Paper. When calcuing the settlement of large buildings on thick layers of clay, buildings in London being case in point, the ultimate settlement was often found to be much more than would be fe; however, the safe limit would not be exceeded for a considerable period. The estion then arising was to what extent the clients should be involved in large capital penditure to avoid trouble in perhaps 50 or 100 years. Of course the answer depended the merits of each case. The situation should be considered carefully before providing pensive foundations to avoid settlements not likely to become serious for perhaps 30 40 years with industrial structures and 50 or 75 years with offices or flats. Most odern buildings were so well built that they would become outmoded in their layout d facilities long before they were worn out structurally. It might be found that when e settlements became serious the owners might be glad of an excuse to replace the ildings with something contemporary rather than being burdened with the amortization large masses of steel and concrete buried in the ground.

*** **Mr F. L. Cassel** (LeGrand, Sutcliff & Gell Ltd) stated that the Authors had ecifically pointed out several times in the Paper that the limiting settlement might be aller than the values put forward, if a building was erected alongside an existing struce and bonded with it.

Records and reports on damage caused in such cases were even more rare than others. their researches had the Authors come across any such reports, though they might ve been insufficient in scope and number to be of value for their purpose?

His question was stimulated by a case where a new framed office building was planned jacent to an existing brick-built warehouse and office building. That had been standing about 50 years and was founded on an alluvial silty clay over Blue London Clay. e strip foundations were said to have been designed for 2½ tons/sq. ft. No settlement nage was known.

The new building would be on piles into the London Clay. For particular local reasons existing boundary wall should, however, be underpinned to the London Clay about deeper, the new foundation being intended to carry both the old wall and the new one. e new foundation loads would be 3½ tons/sq. ft.

The loading and therefore the final settlement under that common boundary wall might kept within the range of limiting settlement for the new building. But what effect ıld the new settlement of the boundary foundation have on the existing building, the of which had no doubt long since settled down to an equilibrium?

Should not in that case the maximum possible settlement also be the maximum differial settlement and how much of it should be allowed?

Dr A. R. Flint (a Lecturer in Civil Engineering, Imperial College) stated that it was prising at first sight that a simple limit might be found to angular distortion of a ventional frame below which no appreciable damage would occur. The correlation ween such distortion and maximum settlement was, in contrast, readily justifiable on obability basis.

n considering the behaviour of a bare frame, or a frame in which the load-carrying acity of the panel walls had vanished as a result of cracking, it might be shown that the

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*** This and the following contributions were submitted in writing after the closure of oral discussion.—Sec.

parameter δ/l did define damage limits within the bounds of Fig. 4. It was supposed that an elastic design basis had been used in the cases cited and that the design working stresses, neglecting settlement, were in each case of the same order. The beam members would be designed to carry bending moments M, proportional to wl^2, where w denoted the loading intensity across the beams and l their span. With regular stanchion spacing the intensity w would be proportional to l in a given class of building. Consequently the section modulus of the beam members would be proportional to l^3. Where differential settlement caused angular distortion of a bay of the frame the end moments M_2 induced in the beams would approximately depend on the parameter $EI\delta/l^2$ where I referred to the second moment of area of the beam. Those moments would be additive to the moment at one end of the beam and would reach a damage limit when the combined moment caused serious distortions or failure of the connexions. That stage would be reached when the total moment was equal to the design factor $F \times$ the design moment, or $M_1 + M_2 = FM_1$. Hence the ratio $M_2/M_1 = (F - 1) =$ constant defined the damage limit, or $I\delta/l^5 =$ constant.

But $I \alpha dl^3$ where d denoted the beam depth and hence the damage limit might be expressed by:

$$\delta/l = \text{constant} \times l/d$$

Since, for economy, the aspect ratio l/d tended to vary little, the basis used in the Paper was valid within bounds set by scatter in floor loadings, relative beam, and stanchion proportions and types of connexion.

Such justification assumed damage to be of the kind mentioned and it would be of value to know the definition of frame damage used by the Authors.

Professor G. G. Meyerhof (Head, Department of Civil Engineering, Nova Scotia Technical College, Halifax, N.S., Canada) considered that the Paper contained the most extensive survey ever undertaken on the behaviour of buildings undergoing foundation settlement.

The limiting amount of angular distortion of traditional buildings was found to be 1/300 for both load-bearing walls and panel walls of brick or similar material in frame buildings, which were liable to suffer structural damage at an angular distortion exceeding 1/150. Both limits were, however, slightly time-dependent because of creep of the materials. For buildings on clays an increase (in the limiting values) from greater creep seemed offset by most of the angular distortion occurring after construction when the structure was more sensitive to deformation than during erection; the reverse was true for buildings on sands. At the less important laboratory loading rates Professor Meyerhof had found[100] much smaller limiting distortions in an analysis of full-scale tests of panel walls and load-bearing brick walls.

Because of the difficulty of predicting angular distortions the Authors' statistical correlations between greatest distortions and both maximum and greatest differential settlements were very useful. Both types of correlations (not only those between maximum settlement and angular distortion) depended to some extent on the effective foundation width, whilst correlations using maximum settlement depended also on the rigidity of the superstructure and foundation or, more correctly, on the ratio of the stiffness of the structure (including foundation) to that of the soil.[100] For foundations on sands the limiting maximum and greatest differential settlements were only two-thirds of those for foundations on clays, but the virtual absence of reported damage of buildings on sands seemed to indicate the conservative nature of the customary bearing pressures on that type of soil.

Allowable distortion and settlement should have a sufficient margin of safety, not only on the limiting values but also against complete failure. Since in laboratory tests failure of panel and load-bearing brick walls occurred at $1\frac{1}{2}$ to 3 times the cracking load[100], Professor

[100] G. G. Meyerhof, "Some recent foundation research and its application to design". Struct. Engr, vol. 31, (1953), p.151.

ieyerhof suggested a minimum factor of safety of 2 on the limiting angular distortion to ısure a customary total factor of safety of 3 for failure of brickwork and similar wall ıaterial. For load-bearing walls where cracking might initiate structural failure, a ıctor of 3 on the limiting angular distortion might be appropriate. To avoid structural amage of frame members, a factor of 1·5 against cracking was generally considered dequate and ensured a total factor of 3 against failure. On that basis the allowable ngular distortions would be 1/1,000 for load-bearing walls, 1/500 for panel walls of brick nd similar unit masonry, and 1/250 for beams and columns of frames.

Since the Authors had shown that settlement calculations for foundations on clays could ıvolve an error of up to 50%, i.e., in the worst cases actual settlements might be 1·5 mes those estimated, Professor Meyerhof suggested a minimum factor of 1·5 on the miting settlements of clays and 2 on those of sands, which were more difficult to predict. n that basis allowable settlements would be 2–4 in. on clays and 1–2 in. on sands, the wer limits applying to small footings and the upper to large rafts; the corresponding reatest differential settlements would be 1¼ in. for clays and ¾ in. for sands.

In view of their statistical approach the Authors rightly emphasized that their criteria f allowable distortions and settlements were limited to tradional types of buildings with ad-bearing walls or with steel and reinforced concrete frames having panel walls of rick or similar material. Little data seemed available for buildings with load-bearing einforced concrete walls or for frame buildings with modern types of curtain and interior alls. In those cases and for special buildings of traditional types the movements which ere tolerable could be decided in each individual case only by structural analysis. In ı earlier Paper Professor Meyerhof had proposed[101] a method by which foundation and ttlement characteristics of the soil could be interrelated with the loading, layout, and gidity of building frames. That method which treated the superstructure, its founda-ons, and the underlying soil as one complete statically-indeterminate system, showed ıat considerable settlement stresses might be induced in all structural members and would e largest in beams at external joints and in all but centre columns, especially in the lower oreys. It was also shown that to prevent overstressing (as distinct from collapse) ermissible bearing pressures or loads had to be reduced as the structure became stiffer.

Since panels and other wall cladding appreciably stiffened exterior building frames, ıat method of analysis had been subsequently extended[100] to give at least a qualitative dication of the behaviour of different types of structures with load-bearing walls or panel alls on different types of soils to estimate the bending moments, shearing, and direct rces in the structural components using actual live loads and simple cases of composite haviour of the building. Whilst in practice the composite behaviour of the structure as more complex, Professor Meyerhof believed that the proposed analysis gave an idea the order of magnitude of the stresses induced by the deformation of structures so that e tensile stresses causing cracking could be estimated and compared with the tensile rength of the building materials to ensure an adequate margin of safety. As a result of cent laboratory research an allowable tensile stress of 30 lb/sq. in. had been suggested[100] r brick walls and two-thirds of the tensile strength for concrete or other building materi-s. As a check of the proposed procedure Professor Meyerhof had initiated the measure-ents at the new Government Offices at Whitehall Gardens, London, to which the Authors ferred. However, many further comparisons between estimated and observed founda-on movements and the corresponding strains and displacements in buildings and other ructures were required to solve the difficult problem of predicting allowable differential ttlements from the calculated stresses in structures.

Professor Karl Terzaghi (Professor of the Practice of Civil Engineering, Harvard niversity, Cambridge, Massachusetts) observed that the Authors' conclusions regarding e ratio between angular distortion and maximum settlement were, in his judgement, too eeping to be accepted in their present form.

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[101] G. G. Meyerhof, "The settlement analysis of building frames". Struct. Engr, vol. 25 947), p.369.

Reviewing the reasoning which had led to those conclusions the reader could not help noticing that, whilst the reasoning in connexion with the settlement of foundations on sand was straightforward and convincing, the statements concerning the settlement of foundations on clay were hedged in by qualifications some of which were rather vague. The reason for the difference was obvious, but the consequences did not receive the attention which they deserved. Therefore, a brief survey of those consequences might be made.

The meaning of the term "foundation on sand" left no margin for interpretation. The term indicated that the entire space between the base of the foundation and the rock surface was occupied by sand and nothing but sand. Experience had shown that the settlement of uniformly loaded areas on natural sand strata varied erratically from place to place. Furthermore the average settlement was practically independent of the thickness of the sand stratum, provided that thickness exceeded about twice the width of the footings. That was because the settlement caused by the compression of the sand located below that top layer was commonly negligible. Hence the relationship between angular distortion and maximum settlement for buildings on sand was reasonably well defined. It was graphically represented in Fig. 4b. However, that Figure could mislead inexperienced readers because it could create the impression that the scattering of the maximum settlement for buildings on sand from the average of the maximum settlement was as wide as that for buildings on clay, shown in Fig. 4a.

All those buildings resting on sand which were known to Professor Terzaghi had settled less than 3 in., whereas the settlement of buildings on clay foundations quite often exceeded 20 in. As a matter of fact the settlement of all the buildings on sand represented in Fig. 4b by points was smaller than 1·2 in. except for one, labelled "96", which had settled no less than 25 in. That was such an extraordinary departure from the average of all the others, that Professor Terzaghi hoped the Authors would give detailed explanation of the circumstances responsible for the abnormal settlement of building 96.

The second point in Fig. 4b indicating a maximum settlement of more than 3 in. represented building No. 82, which rested on "fill". The Authors seemed to take it for granted, that the rules derived for foundations on sand also applied to foundations on "fill" but the meaning of the term "fill" was nowhere explained. Therefore the conclusions pertaining to fills were not necessarily justified. In that connexion Professor Terzaghi drew attention to Fig. 10. Part of the structure represented by that figure rested on a natural sand and gravel deposit and the balance on "fill". About 25 years ago, the fill had been investigated under his supervision. It had been found to consist of an accumulation of brick fragments mixed with topsoil, containing large and irregularly distributed pockets and lenses of a mixture of sand, gravel, clay, and ashes, and of others consisting of city refuse. The relationship between angular distortion and maximum settlement for a random accumulation of that kind had hardly anything in common with the corresponding relationship for a water-laid sand stratum.

In connexion with clay the Authors had emphasized repeatedly that their conclusions were valid only for foundations bearing "directly" on clay. However, they did not exclude foundations which rested on sediments containing lenses of clay within a shallow depth, or foundations which transmitted the weight of the buildings on to the clay by means of pointbearing or friction piles. If a building rested on a sedimentary formation containing erratically distributed lenses of clay, the ratio between angular distortion and maximum settlement might be many times greater than that for a structure which rested on a homogeneous clay stratum with fairly uniform thickness. Furthermore if a building like the Charity Hospital in New Orleans consisted of multi-storied blocks interconnected by light structures with a moderate height, the ratio was many times greater than for structures with a uniform height, covering square areas on the same terrain. Hence the foundations "bearing directly on clay" could easily be divided into many categories depending on the soil profile, the length of the piles and the conditions of point support of the piles; each of those categories could again be subdivided on the basis of the degree of uniformity of the distribution of the load on the areas covered by the building. The corresponding scattering of the values of the ratio between angular displacement and

aximum settlement about the average would be wide indeed. Yet the Authors had ssigned to that ratio a single value, derived from not more than about sixty case records, r probably one to three records for each of the principal categories which could be established. The statistical approach to a problem called for a broader basis.

In 1948, Professor Terzaghi had proposed upper limiting values for the allowable ettlement of foundations on sand (see Appendix II of the Paper and reference 17). hose values were slightly more conservative than the "Design Limits" specified by the uthors under the heading "Factor of Safety". If, in 1948, Professor Terzaghi had nticipated the possibility that an attempt could be made in the future to propose allowable settlements also for foundations on clay, he would have tried to discourage such ttempts by presenting the following arguments.

For clay or artificial fill other than sand-fill the ratio between angular displacement and aximum settlement depended to a large extent on the pattern of stratification and arious other factors such as the distribution of the loads over the area occupied by the ructure. As a consequence the scattering of the value of the ratio from the average for undations on clay was so important that design on the basis of an average could be either neconomical or hazardous, depending on circumstances which were beyond the control f the designer. Therefore a rough estimate of the angular distortion on the basis of the oring records and other data required for predicting the maximum settlement would still e many times more reliable than a mean value derived by some process of averaging from a heterogeneous assortment of case records. The judgement which entered into the timate could materially be assisted by the study of case records, classified in accordance ith the stratigraphic characteristics of the seat of settlement. One might even consider roposing maximum allowable settlements for the different categories of foundations earing directly on clay in individual regions, such as London or Chicago, constituting ell-defined geological provinces, but the number of available case records was still too nall to justify such attempts.

On account of the facts set forth in the discussion, the audacity with which the Authors ad drawn their final conclusions had been a surprise to Professor Terzaghi. Instead of imulating thought and observation in the difficult field of clay foundations, the conusions were likely to have the opposite effect. He hoped, therefore, that the Tables presenting the conclusions would not find their way into text books.

Mr C. F. Ripley (Consulting Engineer, Vancouver, Canada) stated that the most nportant conclusion was the Authors' criterion of angular distortion which stated nerally that the limiting value of angular distortion tolerable without causing cracking wall panels was 1/300, corresponding to a differential movement of ¾ in. in a typical -ft span. The conclusion was very interesting because it corresponded almost exactly the allowable limit of differential settlement recommended by Terzaghi and Peck in 48:

> "Most ordinary structures such as office buildings, apartment houses or factories, can stand a differential settlement between adjacent columns of ¾ inch."

That limit had been adopted widely by soil-mechanics engineers in Canada and was, r Ripley understood, widely accepted foundation-design practice throughout the North nerican continent.

The Authors' correlations between damage by settlement and type of foundation soil ere interesting but could be misleading to those unfamiliar with the mechanics of settleents and resultant damage. Damage to a structure from a given differential settlement angular distortion was a function primarily of the rigidity of the structure, not the type foundation soil. If a building founded on clay would crack and show other signs of mage under a differential settlement of 1 in. between adjacent columns, it would exhibit e same degree of cracking for the same 1 in. differential settlement when founded on nd.

Could the Authors present, in addition to the data in Table 7, a detailed description of

the rigidity of the structures on which the observations of settlement effects had been compiled? The description might contain type of structural frame and flooring, type of exterior wall panelling, type of interior finish, and sensitivity of contained equipment, which sometimes might be more critical with respect to settlement damage than the structure. Accurate descriptions and classifications of extent of damage were equally important. The description might include particular components of buildings affected by settlement, whether interior finish, structural frame, etc., and some indication of the intensity of damage, whether the settlement effects were merely unsightly or caused structural distress together with indication of whether repair involved singular or periodic maintenance or major structural alterations.

Mr Ripley's experience in British Columbia had indicated that there was considerable variation in sensitivity to damage (flexibility of structure) by settlement of the wide range of types of building under construction in British Columbia. His experience and observations were not sufficiently broad, however, to suggest a sound basis for modification and correlation of the Terzaghi limit to various types of structures or classes of superstructures. Buildings of four storeys or less with steel frames, a variety of exterior panel walls, and interior curtain walls with plaster finish, had been successful, using the Terzaghi limit. For buildings with reinforced concrete frames and architectural concrete effects on exterior walls and in the interior, the Terzaghi limit had been reduced. For mill-type buildings, with little or no interior finish and steel frames, the Terzaghi limit had been increased.

The responsibility for the decision of evaluation of the relative rigidity of the superstructure had fallen on the soil-mechanics engineer rather than the structural engineer, in Mr Ripley's experience. That might be right because, first, the limit of settlement was not subject to the methods of analysis of the structural engineer and must be based on judgement or knowledge of effects of settlement in similar structures; secondly, the methods of prediction of settlement of the soil foundation used by the soil-mechanics engineer necessitated considerable judgement and correlations with actual observed settlements, which provided the soil-mechanics engineer with a broader basis of knowledge of allowable settlements. He emphasized that lest the structural-engineer reader thought that the statistical data in the Paper provided a design basis for evaluation both of the probable magnitude of settlements of structures on sand, silt, or clay and of the resultant effects on the structure. In each case, however, the magnitude and pattern of settlement could be estimated reliably only from determined physical properties of the foundation soil obtained by adequate site investigation, and the effects of those settlements on a particular structure would depend primarily on the rigidity of its several components.

Professor R. B. Peck, Dr D. U. Deere, and **Mr J. L. Capacete,*** stated that the Paper rightly drew attention to the necessity for determining the allowable settlement if a rational foundation design was to be carried out.

Numerous factors had to be considered before the allowable settlement for a given structure was established. Although the Authors qualified many of their statements, some of the qualifications should be emphasized and some other aspects should not be overlooked.

First, the highly subjective nature of the evidence on structural and architectural damage, on which the conclusions had been based, should be emphasized. Because no damage was reported did not mean that even rather obvious signs of differential settlement might not exist. In some localities, settlement was not considered unusual and such evidences were treated rather casually. In others, settlement cracks were considered evidence of poor design or contruction, and the designers of a structure would not be inclined to mention them. Indeed, whether or not evidence of settlement was reported in the literature was likely to depend considerably upon the experience of the observer.

* Professor Peck is Research Professor of Foundation Engineering, and Dr Deere is Associate Professor of Civil Engineering and Geology at the University of Illinois. Mr Capacete is President of the Foundation Engineering Co. of Puerto Rico.

he trained observer could probably find cracking in many structures for which no damage as reported but the casual observer might be unaware of signs of differential settlement a well-maintained structure such as the Charity Hospital.

The Authors implied that architectural damage, as evidenced by settlement cracks, was ıdesirable and should not be tolerated. That implication was the reason for attempting correlation between cracking and angular distortion. Nevertheless, there were other ıuses of cracking and architectural damage than differential settlement, e.g., shrinkage, mperature changes, weathering, and vibration. An experienced engineer, responsible r the design of numerous tall structures, had stated that it seemed impossible to construct ı important building without cracks eventually developing. If that was so, it might not sound engineering practice to insist upon restricting the differential settlements to void cracks if such cracks would be no more objectionable than those likely to occur from her causes. Economy, costs of maintenance, and the influence of the cracks on the use the building should all be considered.

For example, in a relatively new apartment house in Chicago, rather moderate differ-itial settlements had produced diagonal cracks in some plastered walls. Under most rcumstances the cracks would be invisible, but the interiors were decorated in modern-tic fashion and many of the walls were painted deep green or dark brown. The finest acks became readily visible as white lines and were highly objectionable to the occupants. nce the apartments were rented for large sums, and the occupants were under no com-ılsion to remain in them, such cracking was serious. However, in a similarly constructed ıilding used as offices, considerably more extensive cracking might be of little conse-ıence. Occasional inexpensive maintenance at the time of decoration might be sufficient eliminate signs of cracking, at least for long periods. To provide an unyielding undation for such a structure, possibly at great cost, did not appear sound engineering. herefore it could be said psychological factors should also be weighed in trying to arrive an allowable settlement.

The Monadnock Block in Chicago, building No. 91, was reported by the Authors to have ıffered both functional and architectural damage. It had experienced a total settlement about 2 ft and a differential settlement of about 5·3 in. Professor Peck, Dr Deere, and r Capacete questioned whether the structure could properly be said to have suffered ıch damage. They knew of no settlement cracks in it, although new supports for the ıvements had had to be constructed, to keep them at a gradient whilst the structure had ttled. A new floor at street level had also had to be added to compensate for the settle-ent. Nevertheless the structure was 60 years old, still fully occupied, and was con-lered a desirable rental property. It seemed highly questionable whether the designers uld be criticized for not establishing it on an unyielding foundation, at a great cost, eliminate settlement..

The Paper mentioned briefly the influence of rate of distortion on cracking. That ppeared to deserve greater emphasis. Those experienced in underpinning were fully vare that a very small differential settlement between columns, produced during under-nning, might cause serious cracking in a building that might have shown few signs of acking despite much larger settlements that had developed during the years.

An appreciable fraction of the settlement might occur before it became of any structural nsequence. In some countries it was customary to erect a building with steel columns ıd concrete floor slabs so that the frame was essentially completed before exterior walls partitions were added. Only the settlement occurring after the panels were inserted uld produce architectural damage. If that was not considered, the allowable settlement the footings might be greatly underestimated, and the allowable soil pressure would be signed far too small a value at corresponding increased cost.

That the extent of damage was not always directly related to the amount of settlement ıd been forcefully illustrated by a project in Puerto Rico involving several essentially nilar buildings which had settled from 1½ to 20 in. The structure that settled 20 in. ffered the least damage and was still occupied. Many of the structures that settled ly 1½ to 2 in. were so severely damaged that they were judged unsafe for occupancy and

were demolished. The magnitude and pattern of the differential settlement rather than the total settlement determined the amount of damage.

The structures were two-storey reinforced concrete apartment buildings approximately 25 ft × 125 ft in plan. One interior and two exterior cast-in-site reinforced bearing walls extended the whole length of each building. Some of the structures were supported by continuous spread footings but the majority were pile supported. The walls rested on grade beams which were supported by two-pile clusters approximately 8-ft centres.

The subsoil at the site was soft organic clay and peat with some interbedded sand layers or lenses overlying stiff to very stiff clay which extended to considerable depth. The depth to the stiff clay varied from zero near the eastern limit of the site to a maximum of 42 ft near the central and western portions. The site was bounded by the sea on the west and the compressible soils and sand were of lagoonal and beach origin. The natural moisture content of the soft organic clay and peat ranged from 50 to 250%. An east-west cross-section normal to the beach showed a rather uniform succession of strata which generally thickened toward the sea.

One of the structures, founded on shallow continuous spread footings, was near the eastern boundary of the site at the outer limit of the lagoonal soils. The eastern portion of the building was underlain by no compressible soil, but the west end was underlain by soft organic clay and peat. Borings taken subsequent to the settlement showed that the thickness of the compressible soils increased very uniformly from zero at the east edge to 18 ft at the west edge. The settlement increased in a similar uniform fashion, from ¼ in. at the east edge to 20 in. at the west. The settlement was appreciable a few weeks after pouring the first-storey walls. Consequently, it was decided to limit the structure to a single storey. Approximately 6 months after construction the measured settlement of the west end was 20 in. A few vertical cracks appeared in the walls but no structural damage was considered to have taken place. The building has been in use for several years although the total settlement was now probably several inches greater than the 20 in. measured following construction.

Another of the structures was supported on piles. The subsoil consisted of approximately 8 ft of soft organic clay underlain by 5 to 8 ft of sand in turn underlain by 10 to 15 ft of soft organic clay and peat. Those formations rested on stiff clay. The density of the sand layer varied from loose to very dense. Many of the piles met such high resistances in the sand that they were cut off without penetrating below the sand. The presence of the underlying soft clay and peat was not known on account of the inadequacy of the soil-exploration programme. Where the sand was of lower density the piles were readily driven through it, through the underlying soft clay and peat, and from 2 to 5 ft into the stiff clay. The arrangement of the clusters of long and short piles under one of the bearing walls was as followed: the first four clusters consisted of long piles, the next five of short piles, the following four of long, and the last two of short piles. As the second-storey walls were being completed, severe cracking was noticed in the walls and floor. Levels were established along the wall and it was noticed that the short piles were settling. The second-storey roof was not poured and the building was observed for several months. The cracking became worse and in less than 1 year after beginning construction the structure was pronounced unsafe and had to be demolished. At that time the total settlement did not exceed 2 in. The settlement under the long piles ranged from ¼ to ½ in. whereas the settlement under the short piles ranged from 1½ to 2 in. The maximum differential settlement was about 1 in. in 24 ft (δ/l equal to 1/288). However, that was sufficient to open cracks ¼ in. or greater in the reinforced concrete walls, to cause failure in bond of the reinforcing steel in the walls, and in one instance to break in tension 1-in.-dia. reinforcing bars in the grade beam.

The preceding comments and examples suggested that the subject of allowable settlements was very complex, and much engineering judgement should be excerised to reach a sound conclusion for a particular problem of design. The Authors were to be commended for summarizing and providing an index to the literature, and for calling attention to the importance of the problem. Some of their conclusions, however, appeared too sweeping,

pecially the implication that a certain amount of cracking, classified as architectural mage, represented unsatisfactory engineering design. It might actually represent the ost suitable and economical solution to a particular problem. If the Authors' limitans on differential settlement were considered inviolate, it would be virtually impossible construct economical commercial buildings in such cities as Shanghai, Mexico City, New Orleans.

Mr W. R. Schriever and Mr W. G. Plewes (Division of Building Research, ational Research Council of Canada) stated that by giving quantitative values for allowle settlement the Authors had made a valuable contribution to engineering knowledge, d were to be congratulated for their painstaking search and analysis of the scattered formation.

The Paper had once more pointed to the importance of field observations. More such ork should be done by collecting and publishing settlement observations and performance cords of buildings.

In making observations of cracks in buildings care should be exercised in distinguishing tween the causes because cracks could be produced by a variety of phenomena of which fferential settlement was only one, although the most important. Certain cases of mage had at first been attributed to settlement but, after close study, had proved to be lated to other causes. One example was the Mount Sinai hospital in Toronto (Building o. 5 in the Paper) where the occurrence of cracks in the terrazzo floors in the hallways was tributed to differential settlement. Fortunately, settlement observations had been ade on the building from the beginning of construction and that theory could be disunted because of the relatively small settlements observed. Further examination owed that the cracking was produced by shrinkage of the concrete floor slabs to which e terrazzo had been bonded. An adjacent building of similar construction showed no ch cracks, where the terrazzo was not bonded to the underlying concrete slabs.

Another interesting example was the cracks in the second-storey walls of the new uilding Research Centre in Ottawa, a conventional steel-frame building with hollow tile alls plastered on the inside.

The diagonal direction of the cracks indicated that some type of racking force was sponsible. Although the effects were similar, the possibility of settlement could be scarded since the building was founded on rock. A few dial-gauge measurements nfirmed the possibility that the cracks might be related to variations in temperature. recorder was therefore installed and provided a continuous record of the crack widths well as the inside and outside temperature of the wall faces. Fig. 13 showed how the ack width closely followed the outside wall temperature.

It was at present planned to study that case more closely. The most probable explanaon was that the roof slab of the building was expanding and contracting in conformity ith variations in outside temperature. That movement applied a horizontal racking rce at ceiling level which at the two ends of the building produced cracks in the end tile nels. In other panels only minor cracks were observed; those were believed to be due flexibility of the window assemblies.

Those observations illustrated the need for distinguishing carefully between damage om settlement and that from other causes. Temperature changes might account for mage to buildings where the differential settlement was relatively small.

It was also worth noting that although the width of the cracks in the hollow tile and aster was close to $\frac{1}{16}$ in. the exterior stucco which was supported on wire lath showed no acks. That was again evidence of the importance of some flexibility between the wallnel materials and the frame. Such flexibility might account in part for some cases entioned in the Paper of large settlement accompanied by little damage. The allowle angular distortion of a given building for either settlement or temperature would erefore depend on the amount of flexibility provided in the design. The Authors had inted out the lack of data on some modern forms of curtain-wall construction that ight have such flexibility.

One American investigator[102] found that, in laboratory tests, the amount of skewing required to produce diagonal tension cracking in a gypsum plaster wall was about 1/1000. The ratio in the Paper was called the "cracking modulus." The larger values observed by the present Authors in the field might result from the flexibility of the connexion between the plaster and the body of the wall or from the difference in the rate of straining.

The Authors had stated that greater distortions arising from settlement might be safe when the foundations were on clay, rather than on sand, owing to the small rates of

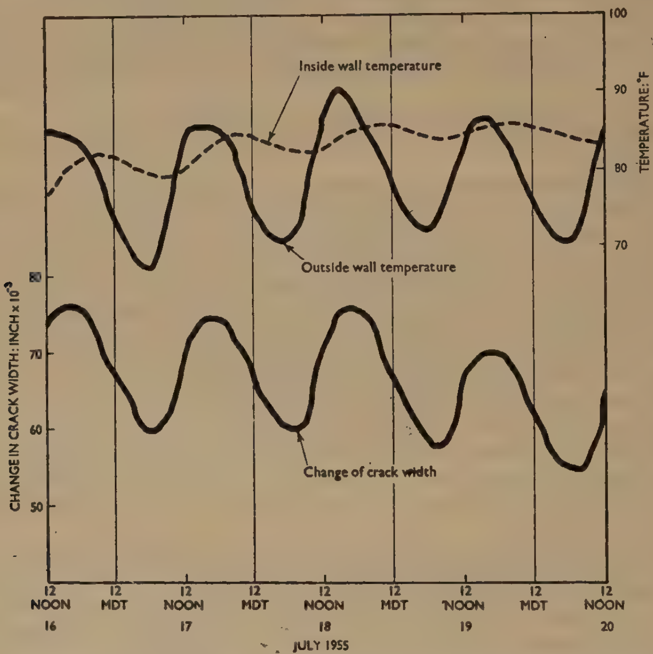


FIG. 13—TYPICAL RECORD OF DAILY TEMPERATURE AND CRACK-WIDTH CHANGES

settlement associated with that type of soil which allowed some relief of stress in buildin materials. It was obvious that movements in buildings produced by daily temperature variations such as those described were of very much faster rate and therefore the allow able values for angular distortion stated in the Paper for settlement might not be vali for temperature changes. It seemed very desirable therefore to have similar field dat collected relating the skewing of frames due to temperature, the length of buildings spacing of contraction joints, and the incidence of damage. Proper consideration c allowable settlement and of allowable movements resulting from temperature change would lead to better building design.

Mr W. H. Ward observed that the expression used by the Authors as the criterion c damage in a building was algebraically:

$$\frac{d}{dx}\left[y-y_0-\left|\frac{dy}{dx}\right|_0 x\right]$$

[102] A. L. Miller, "Plaster cracking as a measure of building motion". The Trend i Engineering, Univ. of Washington, vol. 8 (Jan. 1956).

ich simplified to:
$$\frac{dy}{dx} - \left|\frac{dy}{dx}\right|_0 \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (1)$$

ere y was settlement at any point x measured horizontally, and the term $\left|\frac{dy}{dx}\right|_0$ s the Authors' "rigid-body rotation".

It was difficult to agree with the Authors that the above expression was "only a slightly s logical" characteristic of cracking than the curvature $\left(\frac{d^2y}{dx^2}\right)$. The Authors had licated that in the framed structure stiffened with panels of masonry, it was the panels id the form of their attachment to the frame) which determined when the foundation tortion became excessive. Normally settlement records did not permit local maximum vature of the panels to be calculated, since the settlements were not determined at ervals less than the column spacing, and in that respect the usual settlement records were dequate.

Vhen Mr Ward attempted to make use of the distortion tests on masonry panel walls minally 10 ft square) made at the Building Research Station[104] he was careful not to use results much outside the special conditions and sizes used in the tests. Although only gular distortions had been quoted the frame and the boundaries of the panels became ved and no doubt the onset of cracking was associated with the local maximum vature, the rate of loading, workmanship, and many other factors. Had the panels n larger or rectangular, or the relative stiffness of the frame and the panel been changed, curvature and the onset of cracking would certainly have occurred at different angular tortions.

On a number of occasions it had been observed that panels of identical brickwork of erent size in the same building had behaved quite differently when supported on the ne foundation and subjected to the same settlement. One case, typical of others, ght be quoted. The building was a single-storey gas store for 600 tons of apples built the buttery clay in the Fens. The wall was 18 ft high and 130 ft long; below the uminous damp-course it rested on a further wall 21 in. high supported by a concrete p foundation. The wall was 9 in. brickwork, insulated and sealed on the inside. en the ground floor of the store was first loaded with apples, five vertical cracks eared in the 18-ft high wall. The cracks were widest at damp-course level and ppeared before reaching the top of the wall ;no cracks could be seen in the shallow l below the damp-course and none were visible after subsequent loadings. The settle- nt was dish-shaped in profile although its magnitude was not known since, at the time nvestigation, after three cycles of loading, the store was empty and appreciable uplift he ground and closing of the cracks had occurred. By that time the upper wall had rsailed the lower one by $\frac{1}{2}$ in. at one end. Now the bituminous damp-course was a ne of easy cleavage and clearly the wall (21 in. high) below damp-course level was much e flexible than the main wall above. According to the normal type of settlement rds, had they been taken, both the above walls would have the same curvature and ular distortion at the instant of cracking of the upper wall. Yet the local tensions and vatures in the upper wall must have been more severe.

ield evidence of that type suggested that ordinary settlement records, and angular ortions deduced from them, did not provide sufficient data for determining the criterion racking of a building. Accordingly it was not surprising in the circumstances that the hors obtained a very large scatter of results in attempting to make their correlations he basis of such data.

rofessor Skempton, in reply, wished to thank the contributors to the discussion heir valuable comments. One outstanding point was the subjective nature of the

[104] W. H. Ward and H. Green, "House Foundations: the short-bored pile." Pub. Wks un. Services Congr. & Exhib., Final Rep., p. 373 (1952).

criteria for settlement damage. Professor Peck had dealt admirably with that aspect, and the Authors were in full agreement with him, and with Mr Measor, Mr Williams, and Mr Ripley, all of whom had emphasized that in some classes of buildings (particularly industrial) a certain degree of cracking could, and in some cases should, be accepted, which, in buildings such as hospitals and expensive residential apartments would be inadmissible.

The Authors were aware of that, and of what might be called the "regional sensitivity", on the part of the owners and occupiers of buildings, to settlement damage. It was for that, and other reasons, that the Authors ended the Paper with the words: "It should finally be emphasized that the design limits given above are in no way to be regarded as a set of rigid rules . . . in individual cases the engineer will use his judgement and experience which . . . may lead him to adopt very different criteria." Professor Terzaghi's contribution underlined that sentiment.

In the Paper it had been impossible to cover every eventuality, but the Authors felt there was sufficient evidence to justify their conclusion that it was "unlikely that any settlement damage to a building will occur so long as the limits (suggested in the Paper) are not exceeded." In view of the previous absence of data on allowable settlements for buildings on clay, that conclusion was considered useful, and it was valuable to know that Professor Meyerhof was in substantial agreement with the limits suggested.

There could, however, be little doubt that the most interesting result of the investigations was the limiting angular distortion of 1/300 for panel walls and load-bearing brick walls. That result seemed to have been accepted by those taking part in the discussion. Professor Peck and his colleagues indicated from their example in Puerto Rico that the same value of angular distortion might also represent a limit for reinforced concrete walls. The Authors suggested a limit of 1/150 for damage to the beams and columns of a structural frame (which answered Mr Measor's question). Dr Flint's demonstration that angular distortion was a reasonable criterion for frames, as well as for walls, was an interesting addition to the subject. Mr Ward, on the other hand, had questioned the general application of angular distortion as a criterion for damage. But, for the present, the Authors could see no better criterion which could be applied in practice. In answer to Dr Flint, by the term "structural damage" the Authors implied cracking in reinforced concrete frames and, in steel frames, excessive distortions in the connexions or in the stanchions and beams.

Dr Cooling's question, relating to the causes of excessive settlement, and Mr Ripley's request for a detailed description of the structure of the various buildings, would require further extensive research into the records. But the Authors' general impression on the causes of foundation failure was that such failures usually resulted from the absence of a thorough site investigation. In reply to Professor Terzaghi's query on building [96], the large settlements were believed to result from the bearing capacity of the piles being exceeded. The piles penetrated about 12 ft of sand fill, then about 40 ft of peat and soft clay, and ended in a layer of fine sand, about 15 ft thick, underlain by sandy gravel.

The cases cited by Mr Schriever and Mr Plewes were instructive examples of cracking due to causes other than settlement, and the Authors agreed that in certain cases the results of shrinkage and temperature movements could be confused with settlement cracking. The great majority of the cases of damage quoted in the Paper, however, were certainly due to settlement.

Finally, the Authors wished to thank Mr Souza for his corrections, which had been incorporated in the Paper, and for his examination of the influence of time on allowable settlements. From Mr Souza's data it seemed a correct deduction that allowable settlements suggested in the Paper applied (subject to their various limitations) only on a short-term basis, and that the tolerable settlements for the building and its occupants tended to increase with increasing time after construction. It was hoped that more evidence could be obtained on the influence of time for, as Mr Williams mentioned, that was of considerable practical importance in cities such as London where foundations rested on thick deposits of clay.

Correspondence on this Paper is now closed.—Sec.

WORKS CONSTRUCTION DIVISION MEETING

29 May, 1956

Mr A. C. Hartley, Vice-President, Chairman of the Division, in the Chair

The following Paper was presented for discussion and, on the motion of the airman, the thanks of the division were accorded to the Authors.

Works Construction Paper No. 33

KWINANA JETTY

by

*** Peter Murray, B.Sc., M.I.C.E., and**

Derek Nelson Collett

SYNOPSIS

The Paper describes the design and construction of a jetty built at Kwinana, Western stralia, during 1953 and 1954. The jetty formed part of, and was built concurrently h, an oil refinery for The British Petroleum Company Ltd.

Kwinana is situated in Cockburn Sound, 12 miles south of Fremantle in latitude 32° th. The site conditions and climate are described.

Design.—The layout of the jetty structure was controlled by requirements of naviga- ı, shipping administration, pipework layout and costs, loading operations, and fire- ıting needs. Consideration of these factors led to the adoption of a single shore arm roach and trunkway from which stemmed three jetty heads, each capable of handling classes of tankers up to and including 32,000 tons deadweight.

Because of the need for speedy construction and the highest quality workmanship, the roaches to the jetty heads were designed as an assembly of precast concrete units ich, when placed in position, were connected by post-tensioned prestressed joints to m a continuous structure. Precast concrete construction was also used extensively the jetty heads.

Construction.—Because of site conditions, general design, and the required high rate progress, floating craft were used for most of the construction. Although the major ns of piling and lifting plant were supplied from the United Kingdom several barges l much of the light equipment were built in Australia.

A total of 936 piles were driven including 392 raking piles. All vertical piles were driven n floating craft, positioned from a shore base line.

A precast concrete yard was established on shore, connected to a service jetty by a k-filled bund; in it 1,170 precast units were manufactured. Of these, eighteen end t units weighed 50/70 tons each, the maximum weight of the others being 15 tons. ıcrete attained a high average compressive strength.

Laying and jointing of the precast units are described.

Mixing plants afloat provided 8,000 cu. yd of in-situ concrete.

Severe storms necessitated very substantial craft moorings. Once, part of the per- ıent work and some craft were severely damaged.

The provision of a radio telephone installation was valuable in controlling the works keeping progress at top speed.

A very high degree of co-operation from all concerned was an outstanding feature in execution of the works.

Mr Murray is Joint Managing Director of Kinnear, Moodie & Co. Ltd, and Mr Collett Senior Engineer with Rendel, Palmer & Tritton, Consulting Engineers.

Introduction

One of the major post-war expansion projects of The British Petroleum Compan Ltd (formerly Anglo-Iranian Oil Co., Ltd) was the provision in Australia of a refiner with a capacity of 3,000,000 tons per annum. During 1951 a technical workin party from this company carried out exhaustive investigations in Australia, an eventually recommended that the refinery should be located at Kwinana, Wester Australia. Without delay the necessary negotiations, designs, and planning wer put in hand, and clearance work on the site commenced in October 1952. Th complete refinery, fully operational, was accepted by the company in Februar 1955, nearly 3 months before the scheduled date. The Paper describes the desig and construction of the jetty which was built to accommodate tankers bearing crud oil from the Middle East and elsewhere.

Kwinana is on the west coast of Western Australia, 12 miles south of the port c Fremantle and 25 miles south-west of Perth, the state capital (Fig. 1 shows th site plan). Kwinana is a Maori word meaning "Fair Maid" and was the name of ship which, on fire and abandoned, was blown by a storm on to this shore abou 30 years ago. Until the refinery came into being, the rusting steel hull was th only prominent feature of a low sandy coastline.

Kwinana lies near the southern end of Cockburn Sound, a fine natural anchorag famed among the early navigators around Australia for its safety. The sound protected to the south by a small mainland peninsula and to the west by Garde Island. Between these two is a very shallow unnavigable channel. The sound exposed to the Indian Ocean only to the North-west. Even here protection given by the Parmelia Bank and the Success Bank. These are parallel sandy ba extending from the mainland to the northern tip of Garden Island with a natur depth of water of 10 to 20 ft. The average depth of water in the Cockburn Sound 60 ft and shipping enters from the north-west, through dredged channels in the banks. The tidal range is very small, from 1 to 6 ft.

The beach at Kwinana is of fine sand with occasional outcrops of limestone coral. Wind-blown sand forms a ridge of dunes about 30 ft high parallel to tl beach. Beyond this, flat country extends for 20 miles with vegetation consisting scrub, bush, and eucalyptus forests. Offshore, the sea bed slopes at about 1 in for a distance of about 1,000 ft. Thence it descends rather abruptly to the gener depth of the sound.

The climate is comparable with that of the Eastern Mediterranean. The approx mate extremes of temperature are 42°F in winter and 110°F in summer. Rainf ranges from 32 to 38 in., most of which falls in the winter months, June to Octobe During the summer land and sea breezes are very marked. The sea breeze is tl well-known "Fremantle Doctor", which invariably reduces a noonday temperatu of more than 100°F to about 80°F in less than 1 hour. Winter weather is co ditioned by disturbances about 500 miles to the south passing into the Great Au tralian Bight, and storms are fairly frequent. Cyclonic winds south of the equat are clockwise and the typical gale is from the north or north-west gradually veeri through west to south-west, and diminishing.

Design

Boreholes and test piles

Arrangements were made by the Company for the Western Australian Pub Works Department to put down six boreholes on the site. Two were position

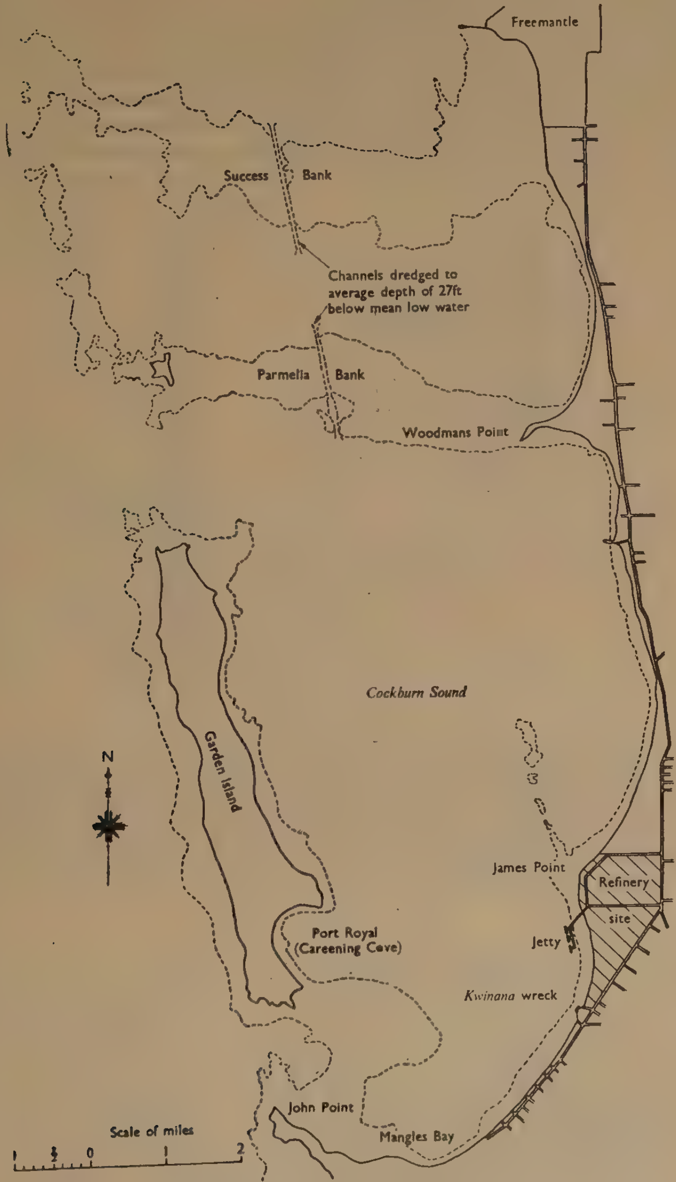


FIG. 1.—SITE PLAN

along the proposed line of the shore arm, and four were put down on jetty-head positions. In addition, six test piles were driven, at the request of the Consultants, in various places on the proposed jetty site.

The boreholes revealed that over the positions of the shore arm the soil consists of layers of calcareous grey sand becoming finer-grained with depth. Over the jetty-head positions it consists of a 20-ft-thick layer of grey calcareous ooze overlying layers of fine white sand, marine limestone, and yellow sand; below the level of — 80·00 grey sand extends to a considerable depth. Typical borehole logs are shown in Fig. 2.

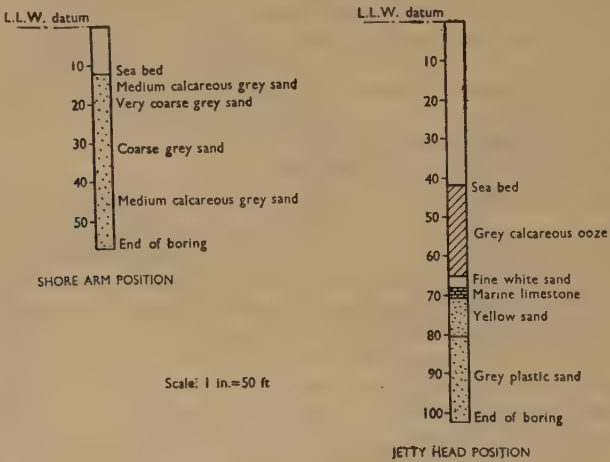


FIG. 2.—TYPICAL BOREHOLE LOGS

The test piles were welded up on site from $17\frac{1}{2}$-in.-internal-dia. steel tubes, an were driven by a 4-ton drop hammer with friction release. After consideration o the results it was found that to achieve the necessary bearing value a penetratio of 25 to 50 ft was required in the shore arm, 55 ft in the trunkway, and 35 to 42 f in the jetty heads. The piles for the structure were ordered on this basis; the desig working load for the piles was 80 tons.

DESIGN DEVELOPMENT

Whilst normal civil engineering requirements of a jetty structure had constantl to be borne in mind during the development stage, the over-riding factor was th need for rapid construction. With a reinforced concrete structure the answ obviously lay in precasting as much as possible of the work on shore so that co struction from floating craft resolved itself largely into pile driving and the assemb of the various units on the lines of a fabricated-steel job. Discussions were initiat with the contractors to determine the maximum weight of lift for the structur units, and after due consideration of the size and availability of handling plant ar floating craft, etc., it was agreed finally that the maximum unit load should b 15 tons. An exception was made for some jetty head units which would requi the use of a heavy floating crane to be hired from the Fremantle Harbour Trus The 15-ton load limitation presented many design problems. The outcome w

nat the approach arms were broken down into an assembly of precast units of hich Fig. 4 is typical. Consideration of the jointing of the units finally resolved self to a choice between an in-situ concrete joint, involving butt-welding of the nain reinforcement, over supports, and a post-tensioned prestressed connexion. n a joint-for-joint basis, the former might well have proved more economical. he second was, however, much quicker because it could perform a "job of work" nmediately on completion of the stressing operation.

A post-tensioned joint was developed using the Lee-McCall system of prestressing. he standard hydraulic jack normally used in this system was considered unsuited nd a more compact jack was specified. Requirements for the design of a new jack lled for a reduction in weight to allow for manhandling around the structure, and r the barrel diameter to be limited to about 6 in. to keep the widths of structural nits to a minimum. Messrs McCall's Macalloy Ltd were approached and a new ck was ultimately designed by them which complied with the Consultants' specifica- on. The weight of the new jack was 82 lb. and a barrel width was $6\frac{1}{2}$ in. compared ith the standard jack at 140 lb. and a barrel width of $7\frac{1}{4}$ in.

On the basis of the preliminary design of the units and their connexions, trial ections were carried out at the contractor's casting yard at Feltham under the pervision of the Consultants. A typical pipe-track-beam connexion, and a com- ete bay of the roadway slab were cast, assembled, and tested. Tests were carried it to destruction on the beam assembly; the road slabs were tested to double e design load. The experience thus gained led to several minor modifications in e design. Valuable data was obtained with regard to handling and placing the its, and the mix requirements for the concrete in the joints between units, and the ethod of compacting them were finalized.

After numerous trials the proportions of the concrete mix for caulking the joints tween units was fixed at 90 lb. : 1 cu. ft : 2 cu. ft. The maximum size of aggregate is $\frac{3}{8}$ in. and the water/cement ratio lay between 0·22 and 0·26 as mixed, the effec- re ratio being about 0·20. The concrete was placed in the joint and well rammed th a mechanical compacting tool; concrete cubes prepared on this basis gave ushing strengths of 900 lb/sq. in. immediately, 7,000 lb/sq. in. at 1 day, and 10,000 'sq. in. at 7 days. Working within these limits it was essential to guard against s or gain of water from the moment the concrete left the mixer until it was mpacted in the joint; the precautions taken on site are described later.

An electric hammer with a frequency of 1,600 blows per minute was found the st satisfactory tool for compacting the concrete in the joint. The hammer had pecial flat-headed bit to fit the width of the joint. To give the top of the joint a ooth closed finish a section of timber, slightly moistened, was placed over the top face of the joint, and a final run was made by the hammer.

General description of structure

The jetty is an open-piled reinforced concrete structure consisting of five main tions—shore arm, trunkway, road necks, pipe necks, and jetty heads. In addition, re are twelve dolphins for securing the mooring wires of the tankers when berthed ngside the heads. A layout of the jetty is given in Fig. 3, Plate 1.

The shore arm and trunkway are jointly 2,550 ft long and carry the pipelines and 6-ft-wide elevated roadway. The roadway slopes down from the trunkway to jetty heads *via* the road neck. The pipelines along the trunkway branch under

the roadway and are carried out to the jetty heads by the trestles forming the pip-
necks. The road and pipe necks are each 139 ft long.

Each jetty head is 235 ft long by 35 ft wide; installed at each end is a system o
flexible fenders to cater for the berthing of tankers. Each head is fully equippe
with hose-handling and fire-fighting equipment together with bollards, access ladders
etc. An aerial view of the completed structure is given in Fig. 24.*

BUND

To economize on piling and construction of a suspended structure to carry th
road and pipetracks, a bund, about 240 ft long, was built out from shore to meet th
jetty shore arm. Random granite blocks, easily obtainable locally, and generall
used in this area for construction of groynes, breakwaters, etc., were specified. Th

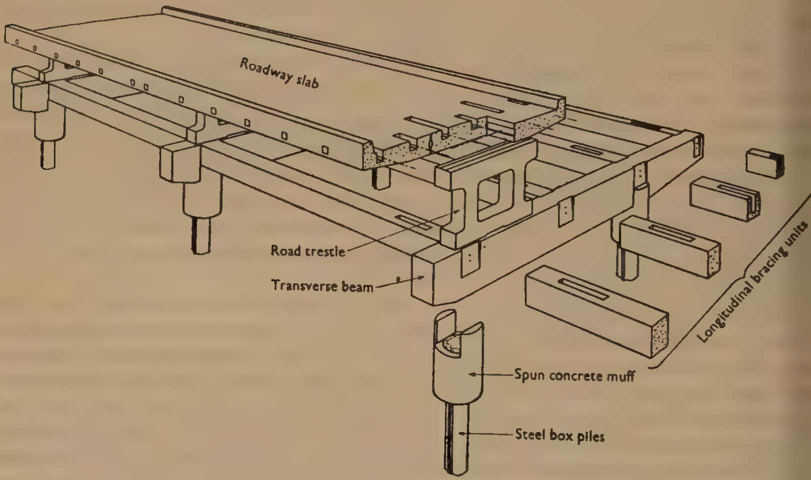


FIG. 4.—PERSPECTIVE OF TYPICAL SHORE ARM BAY

side slopes to the bund were 1 : 1½ and were faced with blocks ranging in weigh
from 2 cwt to 2 tons. A reinforced concrete box abutment, 26 ft long, was con
structed at the end of the bund abutting the road and performed the dual function c
carrying the road at the higher level and acting as a retaining wall. The front c
the box was supported on two steel box piles to guard against a possible slip shoul
erosion of the forward face occur. A nominal hinge was introduced at the base c
the abutment at the connexion with the pile caps to allow for possible settlement c
the abutment at the rear as a result of consolidation of the bund. A cross-sectio
through the bund and box abutment is shown in Fig. 18, p. 809.

PILES

After consideration of various pile sections, notably precast reinforced concret
piles, steel H-section piles, and steel box piles, it was decided to adopt a newl

* Figs 24 to 30 are photographs and appear between pp. 806 and 807.

veloped box section, subsequently known as the "Rendhex" No. 4. Owing, wever, to the urgency of the sitework it was necessary to use a small proportion modified Larssen box pile sections to tide the contract over until the rolls for the w section had been cut and this pile was in full production. To increase the odulus of the normal Larssen section on the YY axis, and to build up the resistance corrosion near the interlock where the wall of the section is thinnest, steel sections ere welded in position as shown in Fig. 5; the Rendhex section is also indicated Fig. 5b.

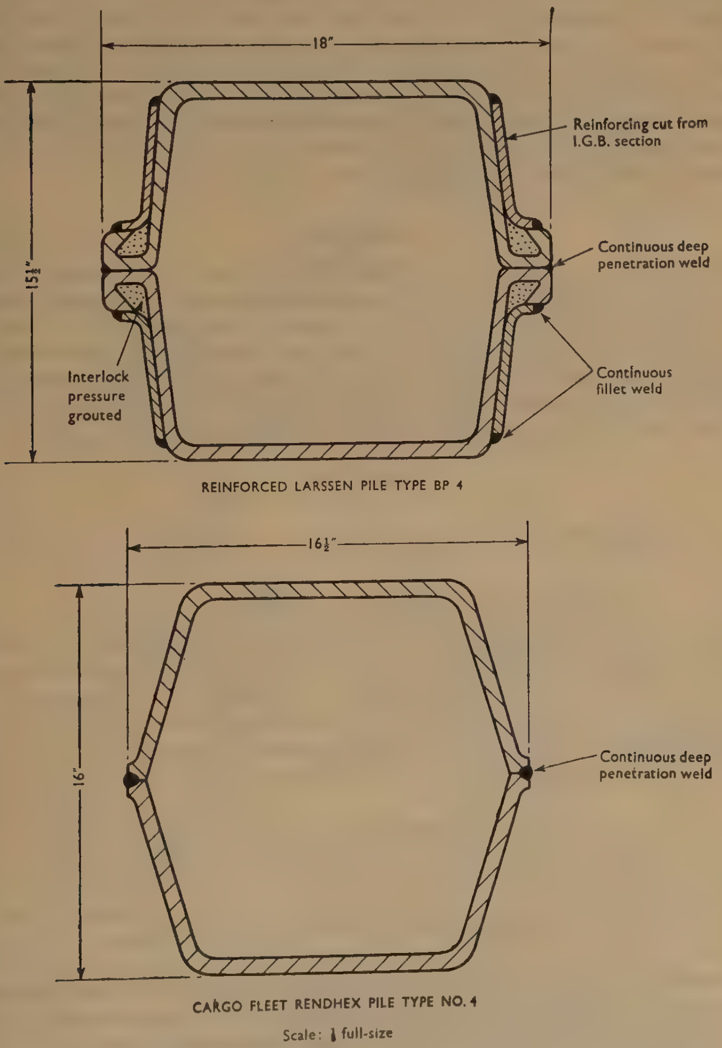


FIG. 5.—PILE SECTIONS

A pointed cast-steel shoe was designed for the Rendhex section and was welded in position to form the toe of the pile. In the case of the Larssen section a chisel-edged toe was formed in welded mild-steel plates. All piles had mild-steel fillets welded to the heads for keying into the concrete deckings.

The piles were manufactured complete in the United Kingdom to the specified lengths, which ranged from 40 to 100 ft, and were shipped to the site ready for driving, except for anti-corrosive paint which was applied on site. Two coats of hot-applied asphaltic-based enamel were used. Details of cleaning the piles and the application of the protective paint are mentioned later.

In addition to painting, cathodic protection was specified for the piles and details of the scheme were agreed in conjunction with the Consultant appointed for the cathodic work. It is not intended to give the electro-chemical details of the scheme in this Paper. Because the approach arms were composed almost entirely of precast units, the bonding of the piles presented certain problems. The means by which bonding was accomplished without materially spoiling the "clean" appearance of the approaches is shown in Fig. 25.

Shore arm

The shore arm consists of twenty-eight pile bents each of 28-ft span; the transverse span between piles is 21 ft. The connexions between the piles and the transverse beams were effected by spun-concrete muffs, of 3-ft diameter, 4 ft 3 in. long, and with a wall thickness of 6 in. The muffs were placed to the correct level over the piles and the annular space was concreted with 1 : $1\frac{1}{2}$: 3 mix concrete. The transverse beam was seated on the muff and grouted in position between the two projecting cheeks as shown in Fig. 6. Fig. 15, p. 808, shows a cross-section.

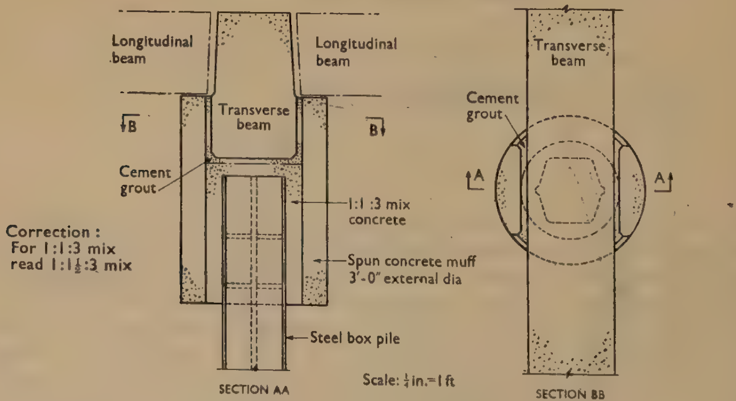
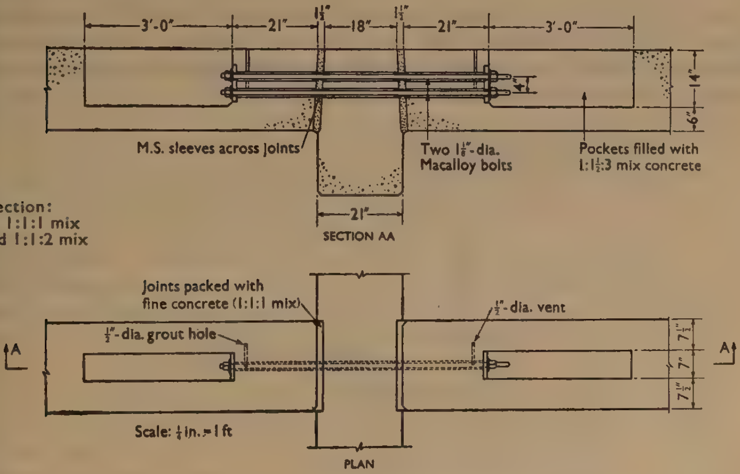


Fig. 6.—Details of pile-muff connexion

The transverse beams are 35 ft 6 in. long, 3 ft deep, and 21 in. wide, and are designed to carry the varying dead and live load conditions which might be imposed upon them by the pipelines and roadways, a check being made for handling stresses. The longitudinal bracing units are 26 ft 3 in. long, 20 in. deep, and 22 in. wide. Normal reinforcement in the bracing units was governed by considerations of the

ading as a result of self weight and handling, in addition to the direct loading esulting from expansion and contraction of the pipelines. The latter force is eveloped at each transverse beam from the friction between the pipelines and pipe ipports and is transmitted axially by the bracing units to the anchor; the force ccumulates from a minimum at each end of the shore arm to a maximum at the onnexion with the anchor slab. To give the correct arrangement of prestress in the ints between transverse and bracing members, two 1⅛-in.-dia. Macalloy bolts were ranged as shown in Fig. 7 to prevent tensions developing from the bending stresses id axial loads in the bracing units.



orrection:
For 1:1:1 mix
read 1:1:2 mix

FIG. 7.—TYPICAL DETAILS OF PRESTRESSED CONNEXION BETWEEN BEAMS

Each of the three precast sections forming the roadway slab is designed as a simply pported unit carrying its own weight between the trestle supports. Under live ad considerations, based on two 10-ton vehicles passing, the slabs were assumed be continuous, and the 1⅛-in.-dia. bolts over the supports were positioned accordgly. The 1-in.-dia. transverse bolts were positioned to give the correct arrangement prestress to prevent tensions developing in the longitudinal road joints as a result loading on the cantilevered wing sections. A typical cross-section through the adway slab is shown in Fig. 8.

The roadway is carried on normally reinforced precast trestles bolted to the ansverse beams by four 1-in.-dia. mild-steel bolts. A number of the trestles in e middle-third of the shore arm were fully fixed, but owing to considerations of fferential expansion between the pipetrack and roadway, the remaining trestles ere given a nominal form of hinge between transverse beam and road slab.

A feature of the joint between trestle and road slab was the use of grout-filled astic tubes. To allow for slight variations in the height of trestles, thickness of ad slabs, etc., 1-in.-dia. plastic (polyvinyl chloride) tubes were filled with grout mediately before placing the road slabs and laid along grooves formed for this irpose in the top of the trestle. The slabs were lowered into position on the estles and allowed to settle on the tubes to the correct level and alignment, where

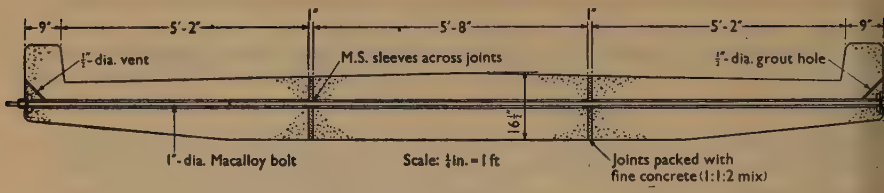


FIG. 8.—TYPICAL DETAILS OF CONNEXIONS BETWEEN ROAD SLAB UNITS

they were held on jacks until the grout had set sufficiently within the tube to carry the slabs. During the caulking of the transverse road joints at the trestle the tube acted also as a seal to prevent loss of the joint concrete. Details of the trestle connexions are shown in Fig. 9.

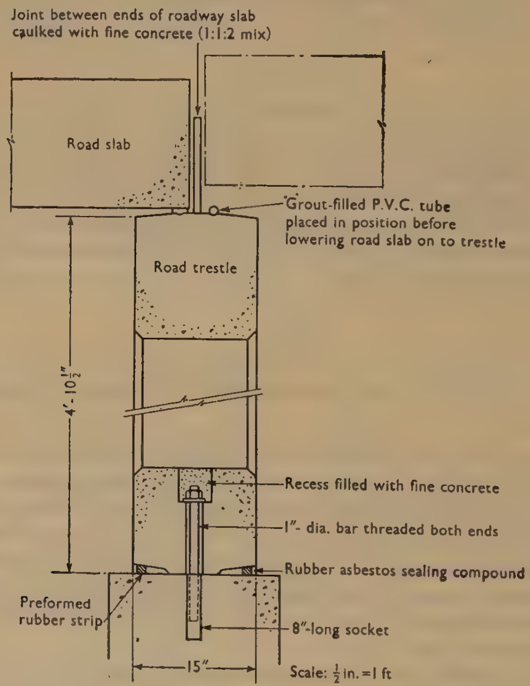


FIG. 9.—TYPICAL DETAILS OF TRESTLE CONNEXIONS

To provide an anchorage for both the shore arm structure and the pipelines an in-situ concrete anchor slab is situated 173 ft out from shore. The slab is 35 ft 6 in. long, 10 ft wide, and 3 ft 6 in. thick and is supported on four vertical and six raking piles. The piles rake at 1 in 2½ and form three A frames in the longitudinal direction. Lateral support is provided at the end of the arm by cogging it into the junction structure in which transverse raking piles were incorporated for the purpose. A 3-in.-wide expansion gap is provided at each end of the arm.

Trunkway

In design the trunkway is similar to the shore arm and consists of sixty-five pile ents, generally of 28-ft span. As a result of junctions with the road necks, however, was necessary to depart from the standard span and introduce a number of 22-ft pans. Because of the more generous spacing of the pipelines necessitated by the iping details at the junctions with pipe necks the transverse beams are 6 ft longer han those in the shore arm and the transverse pile spacing is therefore 26 ft (Fig. 16, . 808, shows a cross-section).

Longitudinal stability and anchorage for the pipelines is provided by an anchor lock situated at the junction with the pipe neck leading to jetty head No. 2. Lateral ipport to the trunkway is provided by the six bollard blocks spaced along its ngth at approximately 124-ft intervals. Each block comprises four vertical and urteen raking piles (rake 1 : 2½) arranged in A-frames in both the longitudinal and ansverse directions, surmounted by a normally reinforced in-situ deck 18 ft wide, 4 ft 6 in. long, and 3 ft 6 in. deep. The design provided for the deck to be cast situ around the vertical piles which were to be driven from floating craft. Open-igs were provided in the slab for the raking piles to be driven subsequently from a ame mounted on the deck of the bollard block.

Twin bollards capable of withstanding 100-ton pulls were cast into the deck. adder access was provided from the trunkway road and precast concrete cantilever rackets were fitted along the front face of each block to carry the timber fendering own to low water level.

Road necks

Three road necks provide access to jetty heads Nos 1, 2, and 3 respectively. The ile bents are spaced at 26 ft intervals and a precast concrete beam or yoke spans etween each pair of piles to provide support for the road slab. The slab ramps own from the trunkway level of + 18·33 to 11·50 at the rear of the jetty heads nd is designed and constructed on similar lines to that for the shore arm. The inction with the trunkway is formed in normally reinforced in-situ concrete and n expansion joint 2 in. wide is provided where the neck abuts on to the jetty head. ateral support is given to the end of the neck at this point by cogging into the tty heads as shown in Fig. 10, Plate 1, and a cross-section is given in Fig. 17a, . 809.

Pipe necks

The pipelines are carried out to the jetty heads on a series of trestles constructed f precast units similar to those used in the approach arms and are connected together a like manner by the Lee-McCall system. The transverse members are 43 ft long nd the distance between adjacent beams is 35 ft. The connexion with the trunkway ipe-track structure is formed in in-situ concrete and two pairs of raking piles are tuated at this point to cater for the loads transmitted to the structure from expan-on and contraction of the pipelines along each neck. An expansion joint is rovided between jetty heads and the ends of the pipe necks (Fig. 17b, p. 809, iows a cross-section).

Jetty heads

Each jetty head is supported on 140 piles, of which seventy-four are vertical and xty-six are raking at an angle of 1 : 2½. The transverse raking piles are at each

end of a head to cater for berthing impacts received normal to the jetty face whilst fourteen longitudinal raking piles are situated in the central portion of the head to cater for the longitudinal component of blows from vessels. Fig. 10, Plate 1, indicates the layout of the piles in a jetty head.

The deck slab is 35 ft in width and has an average thickness of 3 ft 6 in. A cantilever portion 7 ft 9 in. wide runs along the back of each head to carry all service pipes and electric cables. A "skirt" is provided along the front face to house the fender unit fittings and to carry the intermediate timber fenders down to low water level. In addition to normal dead-load requirements the deck is designed to carry a superimposed load of 224 lb/sq. ft.

For speed in construction it was necessary to reduce so far as possible the use of temporary formwork and tidal concreting. Precast soffit slabs and skirt units were therefore introduced. The soffit slabs were generally about 37 ft 6 in. long and 9 in. thick, and incorporated the cantilever portion at the rear of the head. Because of the weight limitation it was only possible to make their maximum width 7 ft. The slabs were designed to carry the full head of newly poured concrete in addition to self-weight requirements and a check was made for handling stresses. The top surface of each slab was specified to be given full construction joint treatment and dowel bars were provided across the joint. It was assumed that after the in-situ concrete had set, the deck slab would act as a homogeneous section to carry all subsequent loads and impacts.

The intermediate skirt units took the form of structural concrete formwork as indicated in Fig. 11 and were designed to fall within the 15-ton lift limitation. The main vertical reinforcement was incorporated in the unit and projected upwards into the in-situ decking. Reinforcement in the longitudinal direction was placed within the $1 : 1\frac{1}{2} : 3$ mix in-situ concrete in-filling to provide continuity across the joint between the units and distribute the loading along the front face of the jetty.

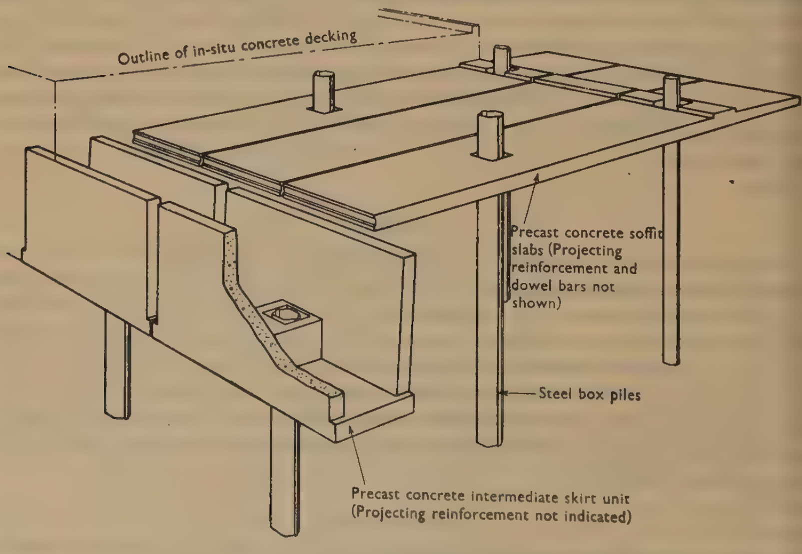
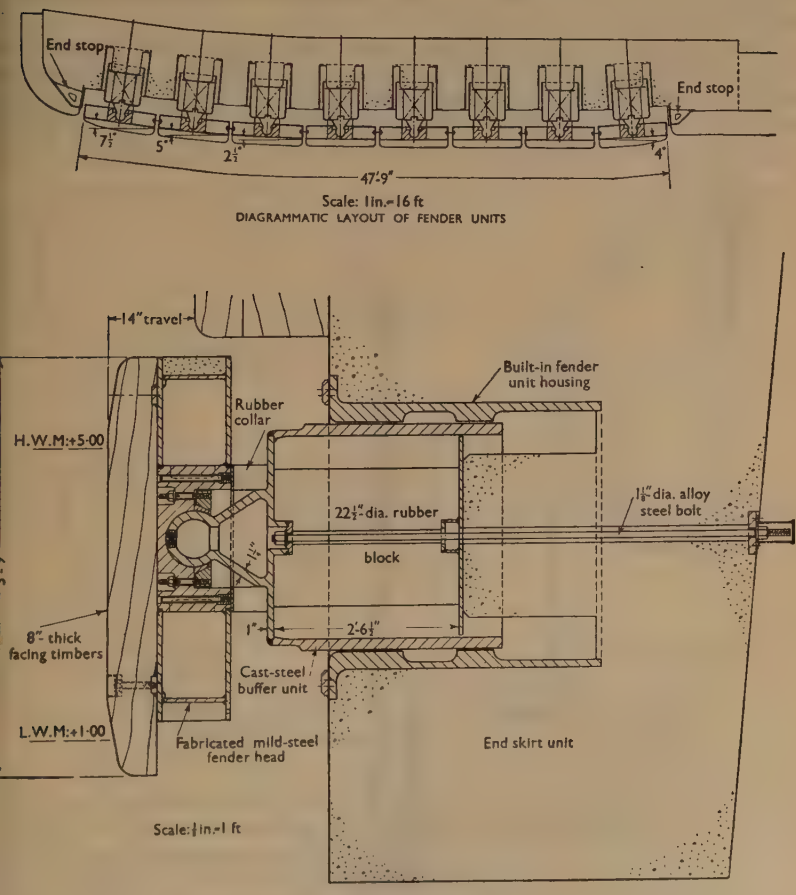


Fig. 11.—Arrangement of intermediate skirt units and soffit slabs

It was not considered practicable to break down the end skirt units to 15-ton lifts, d arrangements were made to hire the Fremantle Harbour Trust's 80-ton floating ane to handle the large units. It was thus possible to eliminate any major tidal ours in the end bays and the units were cast wholly on shore complete with the ilt-in fittings for the fender units. It was, however, still necessary to break down e skirt into three sections, of which the largest was 21 ft long, 8 ft 9 in. deep, and 6-ft average width. Pockets, of 24 in. diameter and 2 ft 6 in. deep, were formed the underside of each skirt to receive the tops of the piles. The units were placed position on the piles by the floating crane and the annular space between each le head and the corresponding pocket was grouted up. A vertical groove 10 in. ep and 15 in. wide was formed in the ends of the units where they abutted each



TYPICAL SECTION THROUGH FENDER UNIT

FIG. 12.—FENDER UNIT DETAILS

other; in this groove a 20 in. × 12 in. B.F.B. was placed. The beam was subse-quently concreted into the groove to form the longitudinal connexion betwee[n] the units.

Before placing the units in position, the details provided for the lower timbe[r] fenders to be fitted in position on shore thereby further minimizing tidal work.

Flexible fendering system

To absorb the energy of berthing vessels a system of flexible fendering wa[s] installed at the ends of the jetty heads. A diagrammatic layout of the syste[m] together with a typical cross-section through an individual unit is shown in Fig. 12 The system was designed to cater for all types of tankers from the small coasta[l] variety up to and including those of 32,000 tons deadweight (approximately 40,00[0] tons displacement). The maximum energy absorption was based on the largest-siz[e] tankers approaching the jetty at a speed of 1 ft/sec.

Consequent upon the small tide range and the rather shallow berthing face to th[e] structure, the fendering had to be as compact as possible, while still being structurall[y] robust, and providing the necessary energy absorption. Therefore a completel[y] flexible face was provided over the critical depth of the contact face for approxi-mately 48 ft, and comprises eight fender units backed by cylindrical rubber block[s]

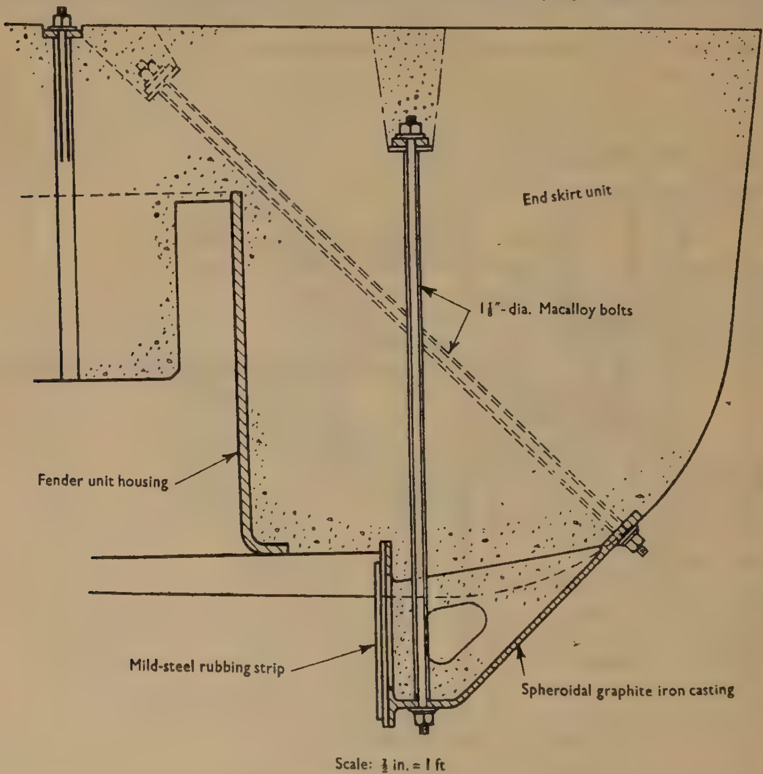


Fig. 13.—Detail of fender end stop

ıe units were connected longitudinally to form a complete assembly and housed ɔo the face of the end skirt units.

The built-in cylindrical housings were manufactured in spheroidal graphite cast ɔn and had an average wall thickness of 1½ in. The fender end stops were also ınufactured of spheroidal graphite cast iron; they were designed to take the ıgitudinal component of a blow from a vessel and were anchored back into the irt unit by 1⅛-in.-dia. Macalloy bolts as indicated in Fig. 13.

The fender units were manufactured in the United Kingdom. Each unit comises a cylindrical cast-steel buffer to which is attached a fabricated mild-steel ıder head faced with 8-in.-thick jarrah timbers. The connexion between the ıffer and the head takes the form of a ball-and-socket joint to allow movement of e head in any direction to take up the vertical and/or horizontal angle of the side the vessel. The fender heads are connected longitudinally by 1⅛-in.-dia. medium gh-tensile steel bolts mounted on flexible seatings to allow differential movement adjacent units.

The cylindrical rubber blocks are 22½ in. diameter and 33 in. long and each block capable of absorbing 600 inch-tons of energy at 50% compression. A typical ad/compression curve for a rubber block is shown in Fig. 14.

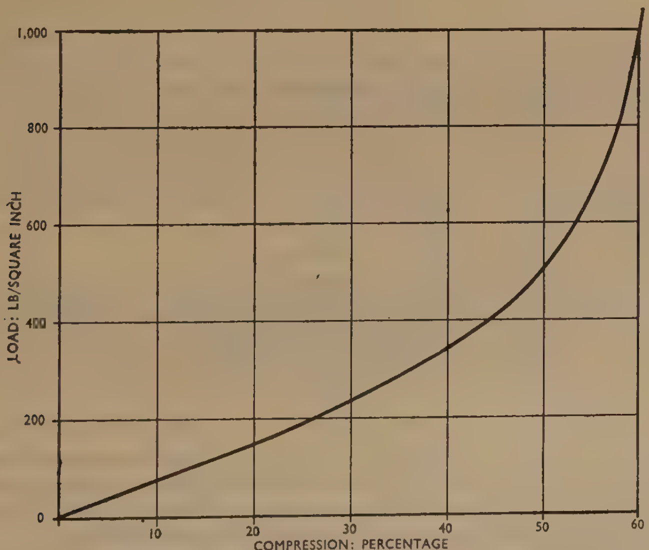


Fig. 14.—Typical load/compression curve for rubber block

The units were zinc sprayed at the manufacturer's works before shipment to ıstralia and were given two coats of bitumen paint before being installed. Cathodic otection was also applied to the units.

Between the main fender systems at each end of the jetty heads protection to the ɔnt face of the structure is provided by 12 in. × 8 in. vertical timber fenders spaced approximately 7-ft intervals. Each fender is faced with an 8-in.-thick rubbing rip. A continuous horizontal waling 14 in. × 12 in. runs along the face of each ad at coping level.

Construction

Planning and preliminary work

During 1952 the Company held joint conferences with the Consulting Engineer and the contractors to consider all matters relative to the construction.

It was clear that the bulk of the heavy construction would best be carried out fro floating craft because:—

(1) The land opposite the jetty was required for the main refinery constructio
(2) The main works, i.e., the three jetty heads, were about 1,400 ft out fro L.W.O.S.T.
(3) The time available for construction was so short that it was essential t work at as many places as possible. Moreover, the movement of heav plant from place to place is quickest and cheapest by water.

It was agreed that the jetty construction yard would be located to the south c the main refinery site, with a short railway connexion to a service jetty. The layou of this yard is shown in Fig. 19, Plate 2.

Selection of plant

As soon as it had been decided that the maximum weight of precast units (excep end skirt units) would be 15 tons, consideration was given to the type and quantit of heavy plant required. It was agreed that the specialized lifting, piling, and floa ing plant would have to be procured in the United Kingdom or in the United Stat of America.

In deciding the type and amount of plant to be procured the considerations physical effort, time, and weather had to be borne in mind.

The physical effort.—This consisted broadly, of:—

(*a*) Preparation and driving of 936 piles including 392 raking piles;
(*b*) manufacture, transport, and placing of approximately 1,200 precast co crete units averaging 10 tons weight including 18 units of 60–80 tons; a
(*c*) mixing and placing of approximately 8,000 cu. yd of in-situ reinforce concrete.

The time factor.—It was essential to the general programme that the jetties shou be completed by the end of January 1955. The need to procure plant in t United Kingdom or America, ship it, and assemble it on site, made it unlikely th much of it would be in operation before July 1953. This meant that only 18 mont including nearly two winters would be available to carry out the work.

There was also an anticipated difficulty in obtaining plant replacements with reasonable time.

The weather.—Although Cockburn Sound was protected from ocean swell, meteor logical records showed that at any time of the year winds of up to 50 m.p.h. mig be expected, and in the winter months gusts of 70–80 m.p.h. had been recorde The site was particularly exposed to the storm winds from the north and north-we and the possibility of substantial loss or damage to the fleet could not be ignored.

Finally, the main lifting and piling plant was agreed as follows:—

Two pontoons each mounting a 65/80-ft piling rig.

Two pontoons each mounting a 10-ton steam derrick.

Three pontoons each mounting a 15-ton steam derrick.
One 15-ton travelling steam derrick for the precasting yard.
One 20-ton steam derrick for the service jetty.

A complete list of all plant used is given in the Appendix.

Preparatory work on the site

Work on the site commenced towards the end of 1952 when the site of the con-ruction yard was cleared and levelled and a gap was cut in the dunes to give cess to the beach. By arrangement, the Public Works Department of Western istralia undertook to construct the service jetty. This jetty was located suffi-ntly offshore to give a natural water depth of 9 ft. It was constructed entirely local jarrah timber, the piles being driven from a small barge with a 2-ton drop mmer and friction winch. A plan and cross-section of the service jetty is shown Fig. 21, Plate 2. The Public Works Department also drove the piles for three icrete blocks to support the 20-ton derrick at the north end of the service jetty. e services of the Public Works Department of Western Australia were particularly luable, because it was unnecessary to send out personnel skilled in piling from the ited Kingdom until the heavy plant had arrived.

The service jetty was connected to the shore by a bund about 450 ft long with a width of 60 ft formed of quarried limestone. This was deposited by a local itractor.

The local engineering industry assisted in the preliminary work much more than l been expected. One firm built four large Philippine barges for the work and the mooring winches. Another firm built one "Fairmile" pontoon and under-k the assembly of two pontoons and several barges which had been shipped from United Kingdom. All mooring equipment, rails, locomotives, wagons, radio ephones, and a variety of smaller plant and equipment were procured within the mmonwealth.

The contractors' key personnel sent out from the United Kingdom for jetty istruction comprised a Construction Superintendent, three civil engineers, sixteen des foremen, and seven heavy-derrick drivers.

They arrived at site on various dates between March and December 1953, and med the core around which the ultimate labour force was built.

embly and erection of plant

The piling pontoons and two 10-ton crane pontoons were made of standard ithwaite tanks bolted together. Each tank measured 13 ft 6 in. × 5 ft 6 in. 5 ft 6 in. deep. The piling pontoons were 61 ft × 44 ft and the crane pontoon s 74 ft 6 in. × 55 ft. All these pontoons had one scow end. The tank sections e assembled on land at the site in blocks, of about 15 tons weight, lifted into the ter, and there bolted together. The decks were suitably stiffened after assembly.

The pontoons for the 15-ton cranes were of Fairmile design and were 70 ft × 50 ft 7 ft with two scow ends. One of these was built complete by Hoskins Ltd, of th, Western Australia, who also assembled the other two which had been shipped m the United Kingdom, each in four sections. This work was carried out on an sting hard a few miles from the site.

The decision to have scow ends on the pontoons was made with some hesitation was perhaps a mistake. The advantages gained in towing for short distances

hardly made up for the difficulties which arose from having too lively a craft in a choppy sea. Otherwise, all the pontoons proved very satisfactory.

The Fairmile barges were shipped from the United Kingdom in two or three self-buoyant sections. On arrival at Fremantle the sections were lowered overboard, and temporarily bolted together. The barges were then towed on to a slipway where the sections were finally welded.

The Philippine barges were 100 ft × 28 ft × 8 ft deep with a capacity of 300 tons. They were manufactured complete by Tomlinson Ltd, of Perth, in their workshops about 30 miles from the site. They were transported by road to a site on the Swan River between Perth and Fremantle, launched, and then towed to the site.

No particular difficulties arose in erection of the derricks and piling frames; this work was much facilitated by the occasional use of mobile cranes from the main refinery site. The first piling rig commenced work in mid-July 1953, 2 weeks earlie than anticipated. With the exception of one 15-ton crane, all the heavy plant was in commission before the end of the year.

Piling

Preparation of piles

The piles were cleaned and painted in the pile yard, situated south of the pre-casting area. Shot-blasting equipment was not immediately available at site an the piles were therefore cleaned by one of the following methods according to th degree of rust and scale present: (*a*) pneumatic rotary wire brushing; (*b*) flam cleaning; or (*c*) pneumatic chipping hammer and wire brush.

The primer used was Wailes Dove bitumastic H.T. grade primer, applied b brush to a coverage of 1 gal/350 sq. ft.

The finishing coat was Wailes Dove H.T. enamel. After a good deal of experimen it was found that the enamel developed the best adhesion when applied at a tem perature between 450 and 480°F. Two methods of application were used, b pouring and trowelling to $\frac{1}{8}$ in. thickness, and by brush.

Brush application gave better adhesion, as evinced by examination of pile extracted from the sea bed. It also involved less wastage (132 sq. ft/cwt agains 98 sq. ft/cwt) but it was very laborious work. In general, therefore, it was decide to apply the enamel by brush to the portion of each pile driven into the sea bed, an by pouring for the remainder of the pile. To protect the enamel during pitchin and driving, rubber-lined rollers were fitted to the deck-level guide on the pilin frames. Damage to the enamel coating caused by welding temporary cleat chafing of wire ropes, and the like was repaired by sandblasting and by the applyin two coats of "Flintkote", a cold-applied asphaltic bitumen compound.

The distribution of piles throughout the work was as shown in Table 1.

The total weight of piles was 4,400 tons and the heaviest pile was 5 tons. A the vertical piles, except two, and fifty raking piles were driven from floating craf The remainder were driven off temporary steelwork framing by the use of a dum frame.

Survey and setting-out

The design of the works generally postulated a high degree of accuracy in positio ing the vertical piles. Moreover, the jetty-head piles were located fully $\frac{1}{4}$ mile fro the beach and it was necessary to commence driving them at a very early stage the work. These two factors made it necessary to pay particular attention to t problem of sighting-in the piles sufficiently accurately. A scheme for building

TABLE 1

	Vertical piles		Raking piles		Total No.
	No.	Length: ft	No.	Length: ft	
ıore arm	69	40–80	10	55–88	79
runkway	137	82	8	86–92	145
hree jetty heads . . .	222	88–93	198	95–100	420
hree road necks . . .	36	82–90	—	—	36
hree pipe necks . . .	32	82–90	8	86–92	40
welve dolphins	48	81	168	86–92	216
	544		392		936

ries of survey stations on timber piles about 200 yd offshore was considered, but as discarded because of the additional time needed and the risk of accidental dis-lacement of the stations by floating craft. It was therefore decided to control all tting-out from a base line on the beach. The base line was about 2,000 ft long and as tied in to the main refinery survey. Stations were established on this line so ıat reasonable triangulation conditions could be obtained by sighting from two ıosen stations on to any pile, the upper and lower limits for intersection angles ɔing fixed at 120° and 60° respectively. The base line was checked and re-checked ıtil the co-ordinates of all stations were thoroughly established. The co-ordinates each vertical pile were then worked out from the setting-out plans. This was the ısic method of sighting the piles but some simplifications were introduced. For stance, in driving the shore-arm piles the centre-lines of piles were projected on to ıe land and only one angular reading was necessary. Furthermore, it was found ıat along the shore arm three bents of piles could safely be measured off by tape, ıd only each fourth bent fixed by triangulation. Later too, a station was estab-shed at the junction of the shore arm with the trunkway and the later trunkway les were sighted in a similar manner. All other piles were sighted by triangulation om the base line. These methods involved a great deal of calculation but it proved be very quick. This was chiefly because of the use of mobile radio sets on the ıore and on the pontoons, and the provision of eight winches on each pontoon hich ensured rigid positioning. The calculation and instrument work was very ɔly carried out, and no errors in positioning were recorded.

riving from floating craft

Each pile-driving pontoon carried the following plant and equipment:—

- One 65/80-ft standard B.S.P. power-operated raking frame.
- One No. 5 Zenith friction winch.
- One No. 22 Spencer Hopwood oil-fired boiler.
- One No. 10 B3 McKiernan-Terry piling hammer.
- One No. 9b semi-automatic piling hammer.
- One steam-powered 5,000 g.p.h. jetting pump.
- Eight 5-ton double-reduction hand winches.
- Eight 1-ton anchors, Admiralty pattern, each with 750/1,000 ft of 7/8-in.-dia. wire hawser.
- One mobile radio instrument.
- One shelter for personnel.

For continuous driving the boiler was found inadequate and a second boiler was installed on each pontoon. The hammers operated at 100 lb/sq. in. steam pressure. Normally the 10 B3 hammer was used because of its shorter height, the semi-automatic being used only for hard driving, redriving of piles for test purposes, and the like. The plant was completely adequate and no particular problems arose in driving either vertical or raking piles. The maximum number of piles driven in 1 week by the two pontoons was thirty-eight.

In general, all the piles achieved a satisfactory resistance when driven to the designed penetration except for the jetty-head piles. Here the diving became very hard and refusal was reached from 2 to 5 ft short of the designed penetration. In many places beds or blocks of limestone up to about 3 ft thick were encountered. These were broken through without much difficulty, although the toe of the pile tended to wander out of position. In this respect the diamond-pointed pile was found to behave better than the chisel-pointed pile.

Driving from fixed positions

The design of the works provided for driving the majority of the raking piles from fixed positions. The piles were driven by a 10 B3 hammer working on a dumb raking frame. A 10-ton derrick provided the lift and steam was supplied from one of the piling pontoons. The dumb frame was adapted from a standard frame, without cat-head and pulley-sheaves. It was 50 ft high and weighed about 8 tons.

In the case of the mooring dolphins the dumb frame was bolted to a temporary steel framework fixed to the four vertical piles of the dolphin, before any concreting had been carried out. On the jetty heads holes had been cast in the soffit slabs, the skirt units, and the in-situ concrete so that the raking piles could be driven after the heads had been substantially concreted.

In both cases, the location of the raking piles involved the careful design of temporary cantilever framework and Fig. 26 shows the set-up for driving the offshore raking piles in the jetty heads.

Manufacture of precast units

Precasting yard

The precasting yard was planned and laid to complete manufacture of the 1,170 units in 56 weeks. Apart from the jetty head end skirts, the volume of the units varied from 7·8 cu. yd to 0·3 cu. yd with an average of 4·5 cu. yd. The casting bed was 850 ft long by 51 ft wide and was paved with 4-in.-thick concrete. A 15-ton travelling derrick commanded the whole of this area and a corresponding stacking area on the south side of its track. Water and compressed air were provided to all parts of the yard by ring mains with suitably placed take-offs. An area to east of the main yard was reserved for manufacture of the end skirts.

Shuttering

On account of the number of recesses, sockets, and other fixings to be incorporated in the units, casting direct on the concrete beds was not practicable and timber soffit slabs were made for all the beams, slabs, and trestles. The faces of these soffits were lined with drum sheeting. The side and end shutters were of $\frac{3}{16}$-in. steel plate stiffened by angle irons. Side shutters were braced across the top by steel angles which also served to hold in position various forms for slots, recesses, and sockets. Side shutters for the deck soffit slabs were of timber as was all the shuttering for the skirt units. Holes for the high-tensile bolts were formed by placing $1\frac{1}{2}$-in.

ternal-dia. galvanized pipes through preformed holes from shutter to shutter. iese were turned slowly during the setting period and subsequently extracted.

inforcement

All the steel reinforcement was of Australian origin and manufacture. It was cut length and bent in the main site steelyard. Reinforcement for all beams and estles was prefabricated in cages and placed by crane into the shutters. For the her units the steel was assembled within the shutters. If necessary, reinforcing eel was cleaned by wire-brushing or by sandblasting.

ncreting

The concreting plant consisted of a 42-cu. yd three-hopper "Stothert & Pitt" ighbatcher, feeding two 14/10 cu. yd "Liner Cumflow" horizontal open-pan xers, one at each end. The aggregate hoppers were filled by a 19 R.B. grab.

Concrete was transported to the beds by four $\frac{5}{8}$-cu. yd Aveling-Barford dump icks. These machines were particularly useful because they could place a full p on the ground and pick up an empty one. Placing was done by direct tipping, shovelling boards, or by mobile crane, according to accessibility. The concrete s vibrated by pneumatic poker-type $2\frac{1}{2}$-in. and 3-in.-dia. immersion vibrators. e top surfaces of all units except soffit slabs were trowel finished and the date casting was marked on each unit.

Curing was commenced immediately after casting by covering with damp hessian. er the final set the hessian was kept continuously wet for 10 days by rotary water-inklers; alternatively a wet sand covering was laid.

The preparation of construction-joint surfaces was a major item and more than 000 sq. ft of concrete had to be so treated. The exposed faces of the deck soffit os were wire-brushed while the concrete was green; this was also done when early king of shutters could be permitted. Nearly half the total area, however, had be done the hard way, by pneumatic tools.

ting

Except for the jetty head end skirts, the units were lifted by strongbacks made R.S.Js with lugs at varying centres to suit the different types of unit. The deck it slabs had to be lifted at three points and a special strongback was made incor-ating a system of wire-rope pulleys and blocks that ensured equal loading. This ongback is shown in Fig. 20, Plate 2.

For initial lifting from the casting-bed use was made, so far as possible, of the ual features of the units as cast. For the transverse beams a group of bolt kets was cast in for fixing pipe supports; for the longitudinal beams and roadway s the bolt holes were utilized. In the deck soffit slabs the protruding steelwork gned for incorporation in the deck slab was suitably positioned and formed in shape of lifting eyes. Once off the casting bed all beams, trestles, and small t units were lifted by slings. The precast units were transported to the service y on 20-ton flat-topped wagons hauled by Ruston-Hornsby D.S.48 diesel loco-ives.

skirt units

here were eighteen end skirt units, six of each of three types weighing 65, 58, 73 tons respectively. Their manufacture provided special problems because

of their weight, the necessary incorporation of the fender cylinders, and the heavy reinforcement around them.

Reinforced concrete casting beds 12 in. thick were laid after the bearing capacity of the soil had been proved. The beds were 50 ft apart to allow space for lifting and loading. The shuttering was of jarrah timber with drum-sheeted faces, and was shored off precast concrete dwarf walls. One end of the shutter was left off to provide access until the unit was ready for concrete. To form the two pockets for the bearing piles, spun-concrete pipes 24 in. internal dia., 2 ft 6 in. long, and 6 in. thick were used. At the top of each pipe a steel plate $\frac{1}{2}$ in. thick was bedded to ensure an even bearing for the unit. The fender cylinder castings were supported in the front by a steel frame erected on the concrete base and behind by suspension from the top of the shutters. The allowable tolerance was $\frac{1}{8}$ in.

The concrete was placed by shovelling from a stage set on top of the shutter structure. Some difficulty was encountered in compacting the concrete, especially beneath the cylinders, and it was found necessary to provide access holes in the shutters through which the vibrators could be pushed.

The average time taken to manufacture the units was:—

Main shuttering	3–5 days
Steel fixing	2–3 days
End shutter and fixing sockets	1 day
Concreting	6 hours
Strike shutters	4 days after concreting
Curing	10 „ „ „
Lifting	14 „ „ „

Anchor bolts were incorporated in the design of the units and special lifting lugs were used. The high-tension bolts anchoring the fender end stops were fixed and stressed before lifting. The units were lifted, by two mobile cranes of 60- and 30-ton capacity from the main site, on to an 80-ton low-loader and towed by tractor to the service jetty. On being lifted the soffit of each unit was very carefully measured, particularly the position and depth of the pile boxes, so that the corresponding bearing piles could be correctly positioned and cut off.

Concrete

Concrete was specified to be not leaner than a nominal 1 : $1\frac{1}{2}$: 3 mix. Grading and the minimum water/cement ratio consistent with workability and thorough compaction were required to be found by trial with the proviso that the maximum water/cement ratio should not exceed 0·45 by weight.

The minimum cube strength requirement was 5,500 lb/sq. in. at 28 days.

Both British Portland and Australian cement was used. The sand was of local origin and very fine. The stone aggregate was crushed diorite of excellent quality and unlimited quantities were obtainable about 20 miles from the site.

The mix adopted for all precast units except the end skirts was:—

Cement	224 lb	
Sand	245 „	
$\frac{1}{8}$ in. stone	170 „	
$\frac{3}{4}$, $\frac{1}{2}$, and $\frac{1}{4}$ in. stone	690 „	(a/c ratio = 5·0)
Water	96 „	(w/c ratio = 0·43)
	1,425 „	= 9·14 cu. ft.

Fig. 24.—Aerial view of completed structure

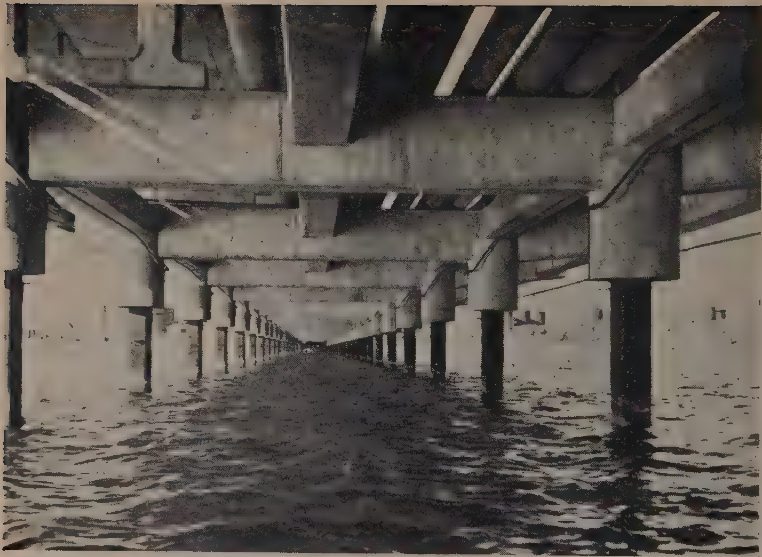


Fig. 25.—Bonding bar along shore arm

Fig. 26.—Cantilevered piling frame driving rakers in jetty head

Fig. 27.—Placing transverse Macalloy bolts in roadway slab

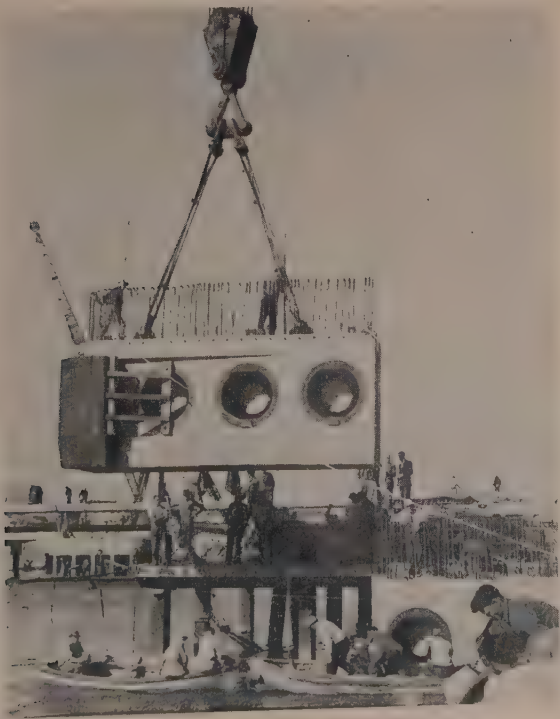


Fig. 28.—Placing end skirt unit in position

FIG. 29.—FENDER UNITS AND RUBBER BLOCKS READY FOR PLACING IN POSITION

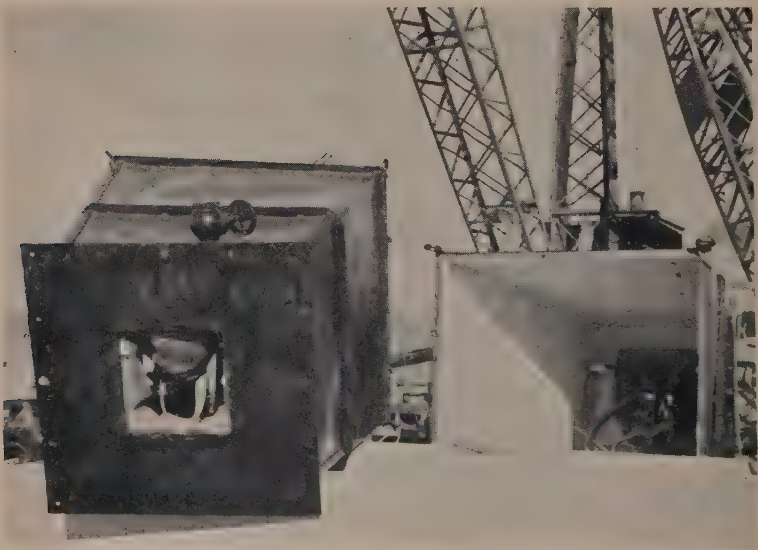


FIG. 30.—COFFERDAM USED FOR PREPARATION OF PILE HEADS SUPPORTING END SKIRTS

In the end skirt units the aggregate/cement ratio was reduced to 4·5 to make the ›ncrete more workable whilst retaining 0·43 water/cement ratio. This ratio was odified only in very hot weather when appreciable evaporation occurred during ixing, transporting, and placing.

The total volume of precast concrete was 5,782 cu. yd and all the units were cast ithin 52 weeks.

The average works cube strength at 28 days was 7,710 lb/sq. in. which was 40% excess of the specification requirement.

Erection of precast units

hore arm and trunkway

The sequence of operations in the construction of the shore arm and trunkway as generally as follows:—

(1) Position the piles.
(2) Heart the piles with plain concrete.
(3) Place and fix the concrete muffs.
(4) Place and fix the transverse beams.
(5) Place and joint the longitudinal beams.
(6) Place and fix the road trestles.
(7) Assemble and joint the road-slab sections.

The work was carried out progressively with three gangs—one positioning the les, one handling and placing the precast units, and one caulking joints, stressing ›lts, and dismantling temporary work. All servicing was done by floating craft.

The piles were positioned by longitudinal steel framing fixed to each row of piles ıd by transverse wire ropes and Sylvester jacks. Despite the absence of any ›preciable tide very little underwater work had to be done.

The piles were hearted from a concreting barge. The concrete was dropped from special bottom-opening skip, straight down the pile, the kinetic energy being con-lered sufficient to ensure compaction. The top 20 ft of hearting, however, was brated.

The pile muffs were seated on temporary timber platforms and concreted to the les immediately after positioning.

The transverse beam was then laid in position between the cheeks of the pile uffs and the connexions were shuttered and concreted.

The longitudinal beams were supported temporarily on bearers suspended from .e transverse beams at the correct level. The stressing-bolt holes of the units were cated by short loose metal sleeves spanning across the gap between the longitudinal ›ams and the transverse beam. The main purpose of these sleeves was to ensure flow of grout along the high-tensile bolts after they had been stressed.

The bolts were then inserted and hand tightened. The gaps between the trans-›rse beam and the ends of the longitudinal beams were shuttered and caulked with y concrete. The mix of this concrete was:—cement 40 lb.; sand 40 lb.; ¼ in. ›ne 80 lb.; and water 10 lb.

This concrete was mixed in electrically driven No. 0 Cumflow mixers. It was aced in the joints in thicknesses of 1 to 2 in. and tamped with an electrically ›erated Kango hammer, with purpose-made tools. Works tests on this concrete owed an average compressive strength of 10,000 lb/sq. in. at 28 days. After ulking, the bolts were immediately tensioned and the annular space around them ıs grouted through preformed holes cast in the unit, by a hand force pump.

The setting of the road trestles calls for no special comment.

Each bent of the roadway consisted of a centre slab and two wing slabs. Each of these weighed approximately 14 tons and they were jointed into a single unit practically in situ. Four stout timber frames 17 ft long were erected transversely, one on each side of each road trestle, and each pair was bolted together to enclose its trestle. The upper edges of the frames when set were about 1 in. lower than the bearing level of the road trestle and temporary timber packings were tacked on to the top edge to a level 1 in. above the road-trestle bearing. The centre slab was

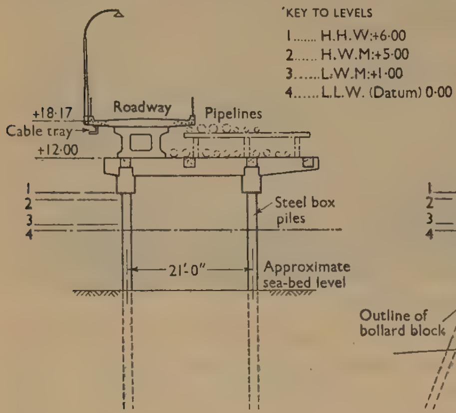


Fig. 15.—Cross-section through shore arm

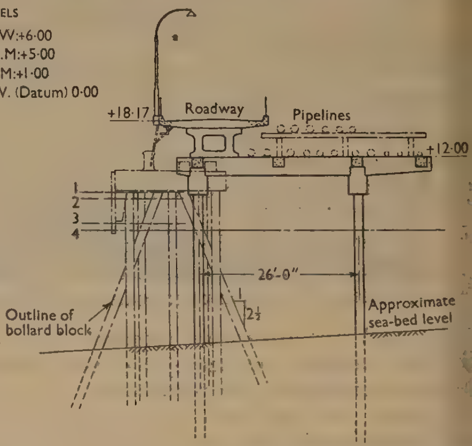


Fig. 16.—Cross-section through trunkway

placed first on the timber trestles in approximately its final position, followed b the two wing slabs which were manœuvred into exact relation with the centre sla by 20-ton traversing jacks based on the longitudinal beams, the longitudinal join between the slabs being maintained by timber spacers. The seven transvers high-tensile bolts were then inserted, the longitudinal joint caulked, and the bolt tensioned. The road slab now weighed 42 tons and was lifted off the trestles b four 20-ton jacks and worked into its exact position in relation to the previou section laid. The joints between each span of roadway were similar to those cor necting up the longitudinal beams.

This form of construction, involving repetition work, lent itself to a high rate progress, and the 15-ton floating derricks with a full-load maximum radius of 82 f commanding four bents, proved to be the ideal tools. The average progress wa about two bents (56 ft) per week and towards the end of the trunkway constructio it rose to the equivalent of more than five bents per week.

In the foregoing construction three types of bolts were used as indicated Table 2.

Jetty heads

The jetty head piles were positioned and contained by a steel framework whic also supported the deck soffit slabs. The front row vertical piles were not include but were left for special positioning to suit the actual dimensions of the skirt unit

TABLE 2.—BOLTS USED

Type	Lengths: ft in.	Diameter: inch	Load: tons	Extension: inch
ams	5 10½	1⅛	45	0·33
ad slab (transverse)	18 3½	1	33	0·76
ad slab (transverse)	4 7	1⅛	45	0·22

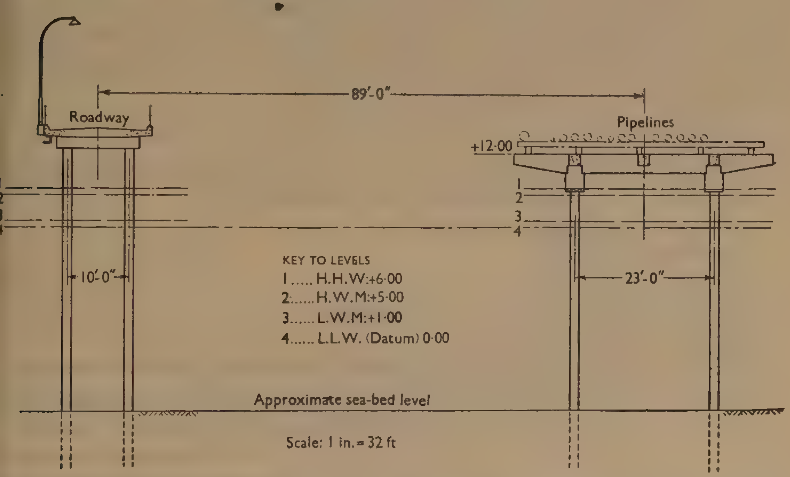


FIG. 17a.—CROSS-SECTION THROUGH ROAD NECK

FIG. 17b.—CROSS-SECTION THROUGH PIPE NECK

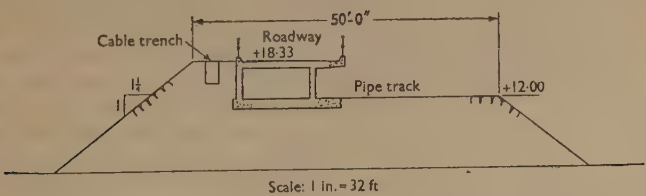


FIG. 18.—CROSS-SECTION THROUGH BUND AND SHORE ABUTMENT

e laying of the soffit slabs commenced at the centre of the jetty and progressed ier way. This presented no special problems. The joints between the soffit os and around the vertical piles were filled with fine concrete and the piles were n hearted. The supports for the sump units were suspended from the main nework.

'he intermediate skirt units were placed as soon as a substantial weight of in-situ k slab concrete had been laid. As with the soffit slabs placing these units was nmenced at the centre of the jetty. The level of the underside of the intermediate

skirt units was + 1·0 ft and was generally under water. The two piles supporting each skirt unit were clamped in exact position by timber framing fixed under water. They were then hearted to within a few inches of the final level and the piles were cut off to the designed level. A small timber platform supported by this framing was clamped to each pile at the level of the underside of the skirt unit. The surface of this platform was faced with a sheet of rubber 1 in. thick, holed exactly to the profile of the pile and fitting it tightly. When the unit was lowered on to the platform its weight compressed the rubber facing to make a watertight joint. Temporary support for the unit was provided by an R.S.J. bolted across the top of each pile box. The unit was brought to a truly vertical position and secured by welding the projecting reinforcement to the projecting reinforcement of the soffit slabs. The concrete joint between the unit and the pile was then made. The concrete joints between adjacent units were made with rubber-faced timber shuttering tightly bound to the outside faces of the units by wire ropes.

End skirt units

The level of the underside of the end skirt units was − 1·0 and the cut-off level o the piles was + 1·5. The preparatory work consisted of:—

(1) Correctly positioning the piles.
(2) Hearting the piles.
(3) Cutting off the piles.
(4) Welding a bearing plate to the pile top.
(5) Welding fillets to the piles.
(6) Welding a bonding-bar to each pile for the cathodic-protection assembly.

The piles were positioned and secured by underwater clamps in a similar manne to that for the intermediate skirt units, and then were substantially hearted. I order to carry out the remaining work in the dry a small steel cofferdam was made This was hopper-shaped 6 ft square at the top, 3 ft 6 in. square at the bottom, an 9 ft high. The bottom plate was holed so that the dam would fit over the pile, an the dam was bolted down on a rubber-faced platform to form a watertight join around the pile. The remainder of the preparatory work was then completed, th bearing plate being kept $\frac{1}{2}$ in. low to allow for adjustments in level by steel packing As work on each pile was completed the dam was removed and fixed to the next on

Thanks to very careful preparatory work, placing the units presented no diff culties. The 80-ton crane collected at a time three units, i.e., one jetty end, fro the service jetty. Two were loaded in barges, the third was laid on the crane dec As each unit was placed on its piles it was plumbed and positioned with the aid Sylvester jacks and secured by welding. All the units were placed to an accura of $\frac{1}{4}$ in. for position and $\frac{1}{8}$ in. for level. The times taken to collect at the servi jetty, transport, and place the six units on each jetty head were: jetty No. 1, 3 day jetty No. 2, 2 days; and jetty No. 3, 1 day.

The underwater grouting of the pile pockets and the concreting of the join between the units were carried out with timber cleats and rubber-faced shutte placed by diver. Before the main deck slab was laid at the jetty ends, the offsho raking piles, previously referred to, were driven through the end skirt units.

General

In-situ concrete

In all, about 8,000 cu. yd of in-situ reinforced concrete was poured. About h was used for pile hearting and the remainder chiefly in jetty-head slabs, dolphi

nd anchor blocks. In the later stages of the work a certain amount of concrete as transported from shore along the completed shore arm and trunkway, but the ulk of it was mixed and placed afloat. There were two floating mixing plants; ach consisted of a Fairmile barge fitted with one 21 cu. yd weigh batcher, two 1/14 concrete mixers, and two 500-gal. water tanks.

The hoppers were serviced by two Philippine aggregate barges each mounting a 9 R.B. crane and grab. The concrete was placed direct in bottom-opening skips f $1\frac{1}{4}$ cu. yd capacity, by floating crane. The output of each equipment ranged om 12 to 19 cu. yd hour.

Each jetty head contained 980 cu. yd of in-situ concrete and was placed in ten o fifteen separate pours ranging from 60 to 140 cu. yd each. Local timber shuttering as used with a face lining of waterproof paper. The permanent fendering, fixed in nal position was utilized as shuttering to the ends of the jetty. A minimum period f 10 days was allowed for curing. The in-situ concrete had an aggregate/cement atio of 4·5 by weight and a water/cement ratio of 0·45. The maximum size of ggregate was $1\frac{1}{2}$ in. Over the whole of the structural in-situ concrete, the works ube compressive strength varied from 5,200 to 7,900 lb/sq. in. with an average of ,400 lb/sq. in.

ssembly of fender units

Placing of the fender units was carried out from the jetty head using a mobile rane and a small gang on a raft. The buffer containing the rubber block and bearng plate was fitted first, being slung in a horizontal position and manœuvred into he cylindical housing. The holding-back bolt was at once fixed temporarily to etain the buffer. The fender head was then fitted to the unit and final adjustments ere made. This work was straightforward.

peration of floating craft

In all, on the works, eighteen operational and service craft were employed, none f which was self-propelled. A considerable organization was required to deal ith the day-to-day movements of this fleet to ensure harmonious progress of the ork. Three vessels were purchased secondhand in Australia after a good deal of earching and were used as tugs. Two were ex-fishing vessels, *Ivanhoe* and *Phoenix*, nd the third, *Manitoba*, was an R.A.F.-type rescue launch. They were not ideal raft; *Ivanhoe* drew 9 ft aft and had a 5-ft propeller whilst *Manitoba* was too light to ow the heaviest barges or pontoons. All, however, were made seaworthy and gave xcellent service within their limitations. Each tug was equipped with radioelephone, which proved an invaluable saver of time. The total number of craft novements carried out by the tugs averaged 33 per working day. Three stout aunches were built at Fremantle and were used for transporting light equipment nd personnel. In addition there were sixteen rowing boats, five of which were ventually fitted with outboard motors.

raft moorings

By arrangement with the Royal Australian Navy six sets of moorings were laid o the west of the jetty and about 1,000 ft out from the jetty heads. These were aid for mooring the service barges and the 80-ton crane; each consisting of two ery heavy clumps connected by ground and trunk chains to a buoy. Experience n the winter of 1953 showed that it was unwise to moor two barges to one buoy,

even in tandem, because of the damage each inflicted on the other in a heavy sea. The doctrine of "one barge, one buoy" was therefore adopted and a further nine moorings were laid down by the contractors. These consisted mostly of concrete clumps 2½ to 3 tons weight made on the site and connected in pairs with ground and trunk chains to buoys as shown in Fig. 22, Plate 2. The final disposition of the main moorings is shown in Fig. 23, Plate 2.

For mooring the pontoons stockless-anchors weighing 16 cwt were used first, but it was found that they would not hold in the sea bed. They were, therefore, replaced by Admiralty-pattern stocked anchors weighing 1 ton which proved much more efficient. The piling pontoons had eight such anchors each with 750 ft of wire hawser. The crane pontoons had four anchors, later increased to five, each with 1,000 ft of wire hawser. All anchors were connected to the hawsers by 30 ft of chain.

During the summer months, long periods of quiet weather were enjoyed with only occasional choppiness in the afternoons due to the "Fremantle Doctor". In the winter, however, conditions were unexpectedly severe and storms lasting for 2 or 3 days would occur at intervals from 8 to 14 days. During the period April to September 1954 no work afloat was possible on 24 days.

The winter of 1954 was indeed an anxious time because the work was at its peak, and of necessity most of the pontoons had to lie to the windward side of the incomplete permanent work. After the whole fleet had ridden out a 3-day storm in May with little damage and no mooring failure, it was decided to run an extra storm wire from each pontoon to a buoy mooring. Also, following a mishap as a result of a crane rope failing in a high sea, all crane jibs were provided with elaborate staying-down tackle. Despite these precautions a minor disaster occurred on the night of 18 July. In this storm a wind velocity of 77 m.p.h. was recorded at Fremantle and a crane pontoon and piling pontoon dragged their anchors. Waves 10 ft high were reported, and since the crane pontoon was impaled by a vertical pile the head of which was at least 4 ft above the water, this seems credible. Both pontoons and a Philippine barge were badly damaged but none were lost and all were patched up. One of the Fairmile concreting barges, however, became swamped, and sank at its mooring. A certain amount of damage was done to the trunkway, and to the soffit slabs on No. 3 jetty head, but the occurrence probably did not delay the works as a whole by more than 2 or 3 weeks. Resulting from this, a second storm wire was carried from each pontoon to a buoy mooring, and although several severe storms occurred subsequently, no further damage was done.

With four or five pontoons moored to the west of the jetty heads the prevention of entanglement of anchor wires presented quite a problem. Distinctive-coloured marker buoys were used for the anchors of each pontoon, and their positions were sextant plotted. By this means, and even more by employing a single gang of experienced seamen to lift and drop all anchors, a chaotic situation was avoided. During 12 months approximately 750 individual anchor movements were carried out.

Labour

Apart from the key personnel sent out from the United Kingdom, all the contractors' labour force was recruited in Western Australia. The peak labour force in March 1954 was about 600 and the average about 340. Labour in Australia is very highly organized and it was necessary to conform to the various basic-wage awards, and to a host of extra rules. The standard week was of 5 days of 8 hours each, and the basic wages ranged from about £13A for a general labourer t

16.10.0A for a tradesman. (£1A is equivalent to 16/– sterling.) It was possible arrange a system of limited overtime at enhanced rates and the working week as, on the average, about 50 hours. A big proportion of the labour force was omposed of European immigrants who would work for long hours at uncongenial sks without complaint. On the technical side several young Australian engineers ere engaged for the work and they proved very able and very enthusiastic, since orks of this magnitude are a novelty in Western Australia.

No strikes or major labour disputes occurred during the construction of the works.

adio network

The radio-telephone installation was supplied by Amalgamated Wireless (Ausalasia) Ltd. It consisted of a base station and eight mobile units. The system erated on a call sign VH 6 BC on 76·7 mc/sec. It is not proposed in this Paper give detailed electrical particulars of the installation. Each equipment consisted a transmitter and a receiver in a single metal case, a power unit, a control unit, aerial, and a speaker. Power was supplied by ordinary car batteries. The base nit was established in the contractor's office. One mobile unit was installed on ch tug and the others were used on the pontoons and other craft according to the portance of any particular work. Mention has been made of their great value positioning the vertical piles. In matters of general control, emergency repairs, aft movements, and the like, the system was of the utmost value in keeping the b at full speed.

Conclusion

In the opinion of the Authors the outstanding features of these works were:—

(1) The great advantages derived from a mainly precast concrete design in building a marine structure, two of which were:—

- (*a*) the higher degree of control over all aspects of concreting which can be exercised in a precasting yard; and
- (*b*) rapidity of construction and corresponding reduction in the hazard to floating craft.

(2) The large-scale application of prestressed joints for connecting precast concrete units.

(3) The flexible fendering system.

(4) The value of radio communication on sea works.

(5) The high degree of co-operation displayed by everyone concerned with the work.

Concerning this last feature, the contractual arrangements were rather unusual. is common in most oil refinery construction, the Company had their own staff progress and inspection engineers on the site. Messrs Rendel, Palmer & Tritton ere the Consultants for the design and construction of the jetty. The main conactors were the Kellogg International Corporation, who were also responsible for e design of the refinery. A consortium of British contractors (Costain–John Brown d, D. & C. and William Press Ltd, and Kinnear, Moodie & Co. Ltd), known as the winana Construction Group, were appointed sub-contractors. This group's duties ere mainly the general administration of the whole works, recruitment, pay, labour ntrol, transport, and so on, under the control of the main contractors. But in dition the group were entrusted with the construction of the jetty and in this they

were technically responsible to the Consulting Engineers. The obvious difficulties which could have arisen by working within such an arrangement happily never materialized.

Thanks to a splendid lead given by the Company's staff the utmost co-operation developed between the British jetty organization and the American refinery organization, and a similar spirit existed between the jetty organization and the Consulting Engineer's staff.

Mention should also be made of the valuable co-operation and enthusiastic assistance given by the West Australian authorities and by local industry.

Acknowledgements

The employers were The British Petroleum Company Ltd, to whom the Author are indebted for permission to publish the Paper. The General Manager, Australia Division (Refineries Department), Mr D. W. K. Barker, M.Sc., and the Projec Superintendent, Mr H. J. W. Braddick, B.Sc.(Eng.), A.M.I.C.E., were responsibl for the project as a whole, and the company's Project Engineer on the site wa Mr C. H. E. Rebbeck.

The Consulting Engineers for the jetty were Rendel, Palmer & Tritton under th leadership of Mr John Cuerel, B.Sc., M.I.C.E. Mr F. I. Childs, A.M.I.C.E., wa Co-ordinating Engineer. The Resident Engineer was Mr J. A. Cole, M.I.E. Aust assisted by Mr J. M. Campbell, B.Sc., A.M.I.C.E., and Mr N. G. Thurstun, B.E A.M.I.C.E.

The Consulting Engineer for cathodic protection was Mr K. S. Spencer, B.Sc.Tech A.M.I.Chem.E., A.M.C.T.

The design and construction of the refinery was carried out by Kellogg International Corporation under the overall superintendence of Mr E. A. Pittalug B.S.M.E., and Mr J. W. Smith, B.S.C.E., respectively. The Resident Manager c the site was Mr G. W. Jones, B.S.C.E.

For the Kwinana Construction Group, the engineer in charge of planning the jett construction was Mr Alexander Cumming, M.I.C.E. (Kinnear, Moodie & Co. Ltd Mr A. T. Dewar, M.I.Mech.E. (Costain–John Brown Ltd) was the Resident Manag in Australia, and Mr G. M. Leslie, B.Sc., A.M.I.C.E. (Kinnear, Moodie & Co. Lt was Superintendent in charge of all jetty construction.

The service jetty was constructed by the Public Works Department, Pert Western Australia, under the direction of Mr N. J. Henry, M.I.E.Aust., Engineer f Harbours and Rivers.

The Authors wish to acknowledge the valuable assistance given by their respecti staffs in preparing this Paper.

APPENDIX

LIST OF MAJOR PLANT ITEMS

Floating plant: tugs and launches

Ivanhoe 66 ft long, 16 ft beam, 9 ft draught. Heavy oak hull. Single screw. Mack & Baxter compound steam engine of 300 h.p.

Manitoba 62 ft long, 16 ft beam, 4 ft draught. Laminated wood hull. Twin screv Two Gray marine diesel engines, each of 165 h.p.

Phoenix 70 ft long, 19 ft beam, 8 ft draught. Jarrah hull. Twin screws. Two Gr marine diesel engines, each of 165 h.p.

Three launches each 35 ft long, 10 ft beam, 3 ft 6 in. draught. Jarrah hull. Single scre Thorneycroft diesel engine 90 h.p.

KWINANA JETTY

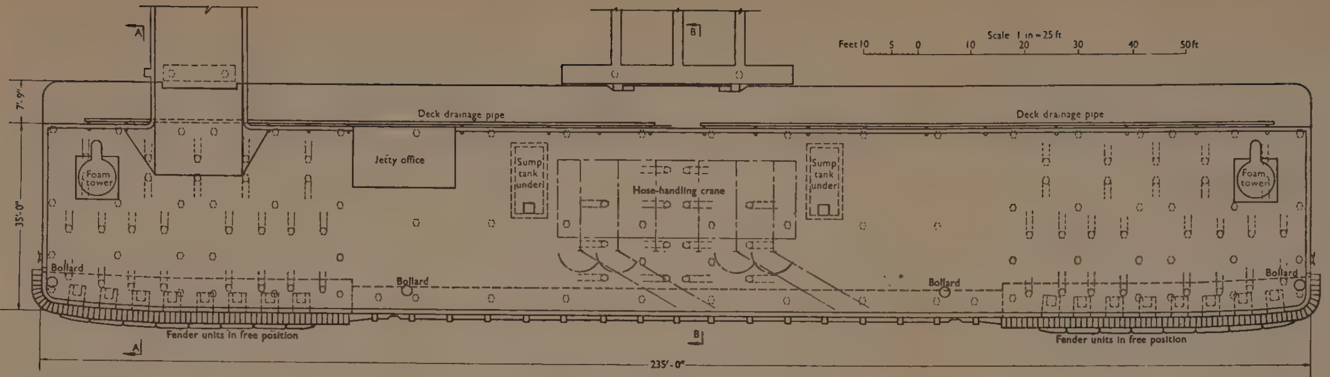


FIG. 10.—PLAN OF JETTY HEAD

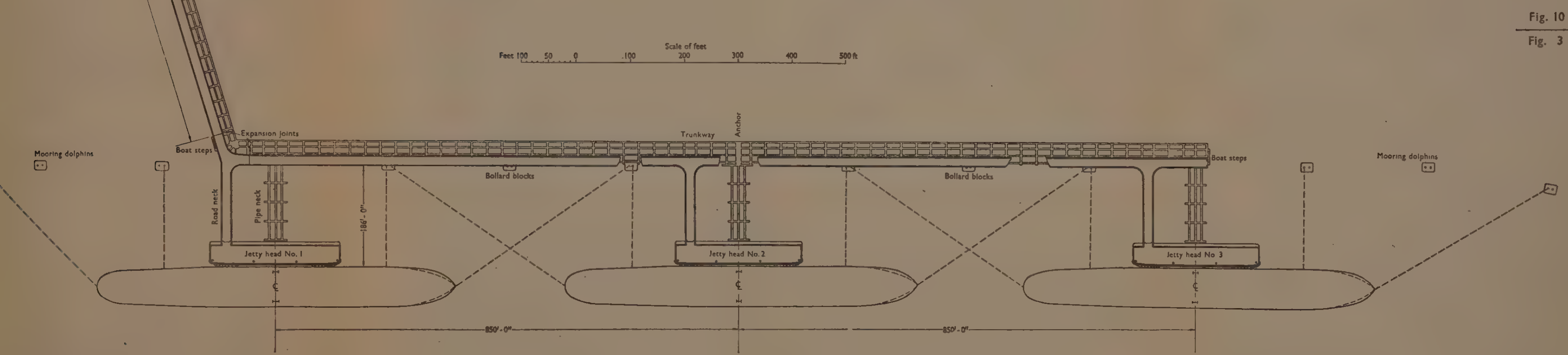
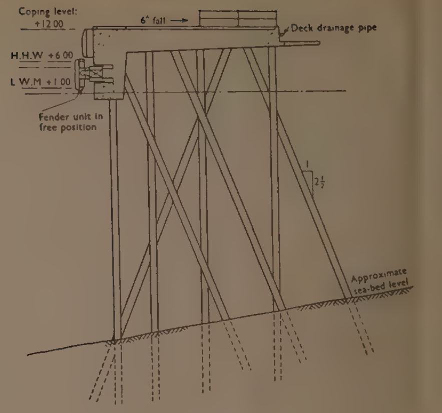


FIG. 3.—GENERAL ARRANGEMENT

Fig. 10
Fig. 3 | Fig. 10



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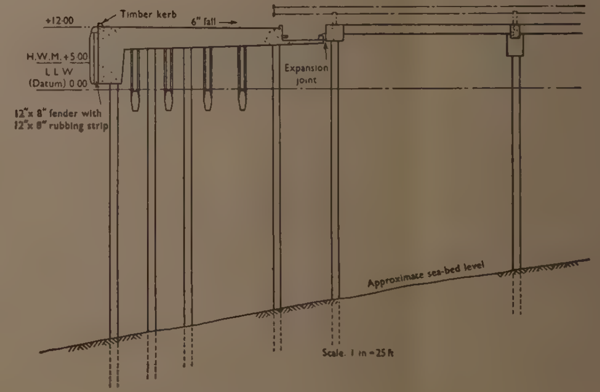

SECTION BB

KWINANA JETTY

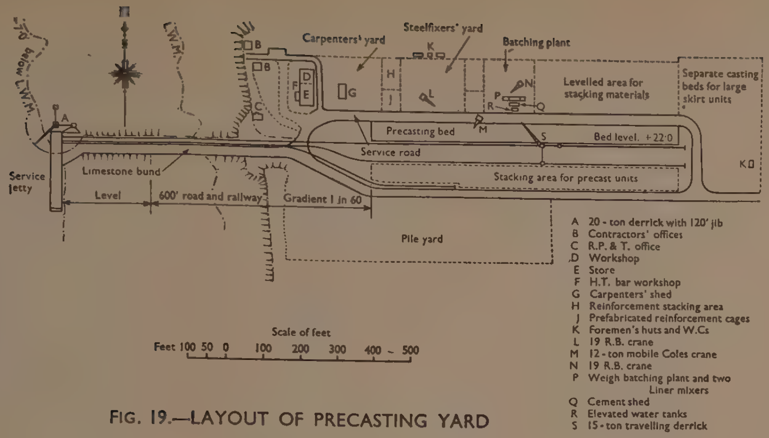

FIG. 19.—LAYOUT OF PRECASTING YARD

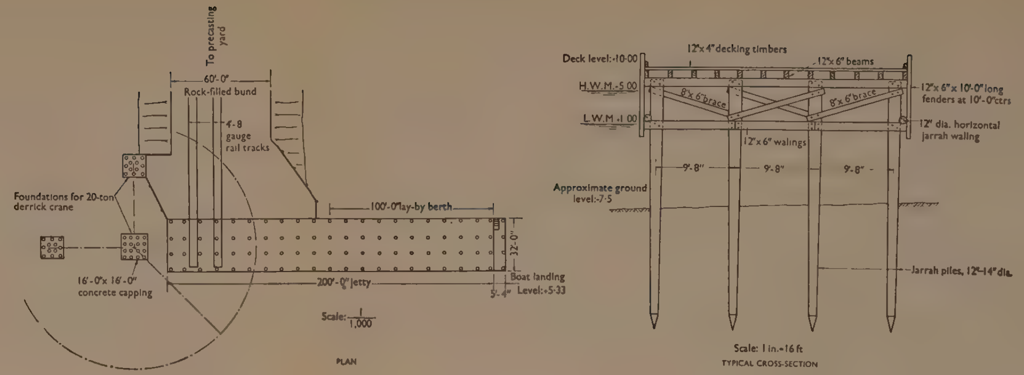

FIG. 21.—DETAILS OF SERVICE JETTY

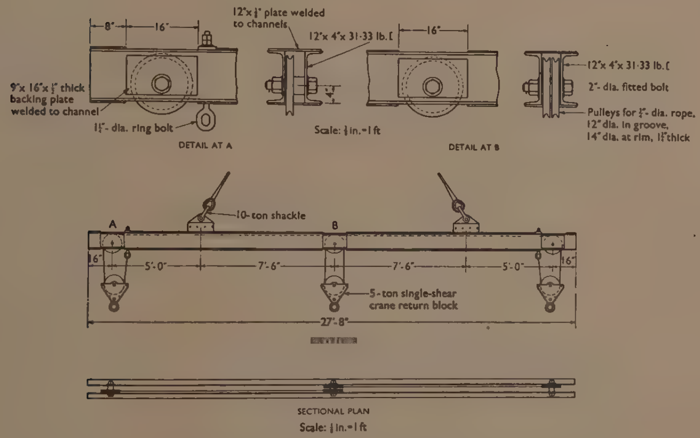

FIG. 20.—DETAILS OF STRONGBACK

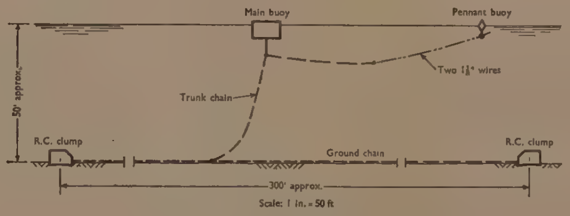

FIG. 22.—DETAILS OF TYPICAL MOORING

Details of moorings

Title	Chain-link diameter: inches	Concrete clumps			Length of ground chain: feet	Depth of water: feet
		No.	Weight: Tons	cwt		
Navy 1	2	2	4	4	500	46
Navy 2	2	2	4	4	500	60
Navy 3	2	2	8	0	360	60
Navy 4	2	2	8	0	360	60
Navy A	1½	2	1	0	360	56
Navy B	1½	2	1	7	360	54
F	1½	2	2	10	360	60
G	1½	2	5	0	320	60
H	1½	2	2	10	320	60
Flume	1½	2	2	10	300	38
Phœnix	¾	2	2	10	320	52
Ivanhoe	1½	2	2	10	300	54
Manitoba	¾	2	2	10	310	50
Spare	¾	2	0	8	160	13
Launch trot	¾	2	0	16	440	10

FIG. 23.—LAYOUT OF MOORINGS FOR CONSTRUCTIONAL CRAFT

ontoons and barges

wo piling pontoons each 61 ft long by 44 ft beam by 5 ft 6 in. deep. Braithwaite tank construction.

wo pontoons for 10-ton derricks. 74 ft 6 in. long by 55 ft beam by 5 ft 6 in. deep. Braithwaite tank construction.

aree pontoons for 15-ton derricks. 70 ft long by 50 ft beam by 7 ft deep. Fairmile construction.

our barges of Philippine construction, 100 ft long by 28 ft beam by 8 ft deep.

our barges of Fairmile construction, 88 ft long by 20 ft beam by 6 ft deep. Hatch 66 ft long, 16 ft wide.

ne barge of Fairmile construction, 120 ft long by 24 ft beam by 6 ft deep, with flat deck.

wo pontoons for water and fuel, 42 ft long by 18 ft beam by 7 ft deep. Philippine construction. Hired from Royal Australian Navy.

ower plant

wo piling rigs, B.S.P. standard 65/80 ft power raking frame, No. 5 Zenith friction winch, No. 22 Spencer Hopwood boiler, 10 B3 McKiernan-Terry piling hammer, 9b semi-automatic piling hammer.

ne piling frame, B.S.P. standard 50/65 non-raking frame.

wo 10-ton steam derricks by Butters Bros. 110-ft jib. Maximum full load radius 82 ft.

hree 15-ton steam derricks by Butters Bros. 110-ft jib. Maximum full load radius 82 ft.

wo concreting plants, each comprising one 21-cu. yd weighbatching plant, two 21/14 or 14/10 concrete mixers. Mounted on Fairmile barges.

hree cranes (two RB 19, one RB 22) mounted on Philippine barges.

lant ashore

ne 20-ton steam derrick by Butters Bros. 120-ft jib. Maximum full load radius 90 feet Located at service jetty.

ne 15-ton steam travelling derrick by Butters Bros. 110-ft jib. Maximum full load radius 82 ft. Located in casting yard.

ne 12-ton mobile Coles crane.

ne 42-cu. yd weighbatcher by Stothert & Pitt.

wo 14/10 Cumflow concrete mixers by Liner Machinery Co. Ltd.

ne RB 22 diesel grab crane.

our Aveling-Barford dump trucks 20 cu. ft capacity.

Two D7 side-boom tractors.

One International tractor model AW6.

wo Ruston & Hornsby diesel locomotives type D.S.48.

Three 5-ton lorries.

Air compressors, portable welding sets, diesel lighting sets, and items marked * were applied by the main refinery site to suit requirements.

The Paper, which was received on 24 February, 1956, is accompanied by twelve hotographs, and twenty-three sheets of drawings, from some of which the alf-tone page plates, folding Plates 1 and 2, and the Figures in the text have een prepared.

Discussion

Mr H. J. W. Braddick (Superintending Engineer, Eastern Refineries, British etroleum Co. Ltd) said he wished to amplify what had been said about the need for speed construction, which he thought had influenced the type of design chosen.

His company had planned a refinery in Australia some time before the Persian troubles ut, with the loss of the refinery at Abadan, there had, of course, been an urgent need to btain additional refining capacity at the earliest opportunity. But it had not been just

as simple as that so far as Kwinana was concerned. It would be appreciated that wher any new refinery was commissioned there was a major change-over in the shipping anc marketing arrangements covering the area served by the refinery. In cases such as Ader and Kent, it was possible to absorb any last-minute changes by redirecting tankers tc other refineries in the vicinity. In the case of Kwinana, however, because of its remote ness from other centres, it had been necessary to plan some considerable time ahead the final commissioning and completion date.

The jetty formed part of an overall refinery job. So far as the refinery itself was con cerned, it had been known that the subsoil foundation conditions involved no majo problems. The jetty, on the other hand, in common with most marine structures, wa always liable to the vagaries of nature, and, as had been seen from the film, the site staf had had their fair share to contend with. It was to the credit of the Authors—and tc his own company's civil engineering staff, who had co-operated to the full in the early stages of planning and development—that the whole job had been completed in accordanc with the planned date, which itself was 3½ months ahead of the original programme.

There had been criticism of the fendering arrangements. The jetty had now been ir operation for about 18 months. Tankers had been damaged—the film had shown the sor of weather that was encountered—but there was no doubt that much of the damage hac arisen from the inexperience of the local pilots in handling tankers in those waters. H had heard the view expressed that the 14-in. travel allowed in the fender units was no sufficient for the conditions and that the fender units themselves should preferably hav been taken to deck level. The coming winter period would indicate to what extent th pilots were at fault and to what extent the rubber units which were referred to by th Authors would assist.

In conclusion, he added that the fender units were designed to absorb blows betwee the ship and the shore, and in his experience they had also to absorb the differences o opinion between the marine and the civil engineers' points of view.

Mr John Cuerel (a Partner in the firm of Rendel, Palmer & Tritton) observed tha the Paper referred to the dry concrete caulking of the joints. He showed a slide of th caulking of the transverse deck joint. That method of making joints had been in use fo about 30 years, but with the development of prestressed concrete and the rapid advanc of precasting in general, emphasis was being placed on some method of forming a join very rapidly which was densely filled and sound in every detail. The joints used on th Kwinana job met those requirements. There was nothing inherently difficult abou making them, but when they were required to be a production job, as in the present case it called for great thought and care in getting all the details correct. The trial run carried out at Feltham had been invaluable in indicating the right tools and procedure t be adopted. They had also shown the complete soundness of the filling and the intimat contact of the joint concrete with the precast faces and had illustrated quite remarkabl the way in which the concrete, although being hammered downwards, swelled up an filled under bars and ducts crossing the joint. Hammered concrete was, of course, ver sensitive to water content. Mr Cuerel thought that perhaps the Authors might hav stressed a little more that it was the effective portion of the water/cement ratio whic counted, and that allowance should be made for losses by absorption by the aggregate and by the joint faces and by evaporation. It was not made clear in the Paper that th figures given for the water/cement ratio covered those losses; the allowance should b fairly close to the mark in order to get an effective ratio of about 0·2, which permitte full compaction by hammering without at the same time getting squashy.

In describing the fendering system, the Authors had outlined the energy-absorption re quirements. Those, in conjunction with limit stops, were all right so far as the fender unit themselves were concerned; but Mr Cuerel considered that for an important tanker jett taking big ships it was necessary that there should be an adequate reserve of strengt in the structure for the occasions when things did not go according to plan, and h thought that the aim should be an ultimate resistance of about 1,000 tons at each end.

The Paper referred to the permissible erection lift for the precast sections of the worl

'he fact that the job had gone ahead rapidly and had been finished before time indicated hat the decision to make a general limit of 15 tons was not wrong, but he thought that he permitted exceptions were more interesting.

Mr Cuerel concluded by showing slides of the skirt units; the first showed them in the asting yard, the second the transportation of a unit to the temporary loading jetty, the hird and fourth the final lowering of the sections into position.

Mr R. M. Wynne-Edwards (Managing Director, Costain–John Brown Ltd) said that he Authors had referred to speed of construction controlling design almost as if it were nusual. Surely, to construct civil engineering works quickly so that the owner could egin getting returns on his capital outlay as soon as possible would always be of primary onsideration.

He had visited the Kwinana jetty two or three times during the course of its construc- on and had been impressed by it being precision engineering on a large scale. When Ir Cuerel had made up his mind to use precast units he had resolved on a complete jetty f precast units, which meant that everything had to be accurate within much finer olerances than were commonly used in civil engineering work. To achieve that required ery nice judgement. The contractors who had had to carry out the work had quite ghtly been brought into collaboration at an early date and the experiments at Feltham ad been carried out. But Feltham was a long way from a jetty to be built in the sea in Vestern Australia and Mr Cuerel had had to judge how accurately the work would in fact e done on the site.

As the slides which the Authors had shown had demonstrated, the piles on which every- ing depended were in fact driven "spot on". The precast units were also fine achieve- ents. The deck slab for instance, where each bay was made up of three units weighing 4 tons each, was tied together with transverse bolts threaded through from one side of e deck to the other. Plane faces were in fact plane and bolt holes matched exactly. s a result, when the time came to assemble all the parts together the work had gone noothly and fast.

Mr C. H. E. Rebbeck (Project Resident Engineer, British Petroleum Co. Ltd) remarked at, as the representative of the client in Australia for the building of the refinery, he uld say that the design and construction of the oil port had been completely successful.

When it was appreciated that as late as June 1952 it had not been finally decided hether three jetties were to be built, or the shore arm, the trunkway, and the three jetty ads, and that at the end of 1954 the first tanker was pumping crude oil ashore, he ought it lent a certain colour and romance to the job.

In common with the rest of the refinery construction, a decision had been made early to incorporate a high degree of tooling for the job. One reason for that had been the cessity for speed. Another very cogent reason was that labour in Australia was ex- emely expensive, so that the less the manual work in a job the more economical it would , provided that the plant was used to the best advantage. He could give an assurance at in the case of the Kwinana jetty that had been so. The job had not only been mpleted ahead of schedule, but also within the estimated cost.

Nearly 1,000 piles had been driven and more than 14,000 cu. yd of concrete had been ced within 17 months, with a peak labour force of only 600 men. Of those 600 men, irty were specialists sent out from the United Kingdom.

The high degree of planning and of direction which that effort represented was self- ident; but it was interesting to note that the work had been carried out concurrently th the construction of a large refinery close by, and also a new township had been in urse of building about 2½ miles away. The State Government Departments had been ry busy on schemes to bring water to the site and to widen the roads. The project d been carried out at a time of over-full employment and in an expanding country, ich in any case suffered from a lack of skilled artisans.

Mr B. F. Saurin (Senior Civil Engineer, British Petroleum Co. Ltd) said he had been ry interested in Mr Wynne-Edwards's remarks about the speed of construction and the

emphasis on speed in America. Those remarks had reminded him that people who had gone from Britain to America to find out how the Americans did things quickly in the building and construction world had come back and said that there were three essentials; the first was that the client or employer must know exactly what he wanted, the second was that very detailed planning in the early stages must be undertaken (planning not only of the main structure but also of the installations and equipment associated with it), and the third was that there must be no alterations.

Those were, of course, counsels of perfection, but since it had been hoped to make Kwinana the perfect refinery, it had had to have the perfect jetty and an effort had been made to follow those counsels of perfection. In the first place, the client had really tried hard to make up his mind what he wanted. That had not been too difficult because, of course, the Company had built several jetties before and several new refineries since the 1939–45 war. So it had been possible to write a design specification for the information of those concerned saying what the Company wanted in the way of berthing facilities, mooring facilities, pipelines, road access, and so on.

It would be noted from the Paper that, associated with each of those functions of the jetty, there was a separate part of the structure—the jetty head, the dolphins, pipe tracks, etc. There had been consideration of how those elements could best be combined to achieve economy in construction and fitting out, and also to suit the natural character of the site. Soundings and borings had been put in hand in order to get all the information available ready for the detailed planning, which was the next stage.

At that point the consulting engineers had been engaged and had been brought into collaboration with the refinery designers and the jetty contractors, and with their advice a final choice of the layout had been made. Then they had set to at weekly meetings to thrash out the details of all the installations—electrical installations and pipe-lines were the major items, with their associated valves and fittings. With so many precast members to be made, it was easy to imagine the trouble there would have been in drilling hundreds of bolt holes and fitting on clips, etc., if full particulars of the fittings had not been available when the precast units were drawn and their reinforcement detailed.

It was at that stage that the difficult decision had been made about the maximum lift. Mr Saurin thought that everybody concerned would have liked to have gone beyond 1[illegible] tons, but circumstances had been against them.

Schedules had then been drawn up of the design work required, the plant needed, etc. After that had come the detailed design and the preparation of the working drawings.

The risk of alterations for technical reasons had thus been minimized and since the circumstances of the oil industry had permitted the Company's policy to be maintained throughout the period there had in fact been no alterations.

The progress schedule of the work (see Fig. 31) was worthy of notice with a view t[illegible]

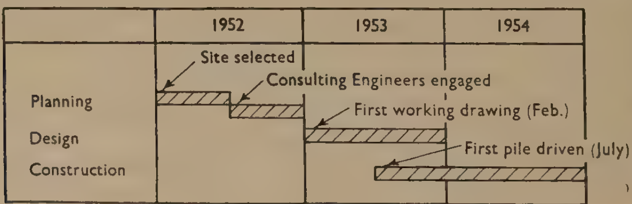

Fig. 31

assisting similar work in the future. At the end of 1951 the first Company man had se[illegible] foot on Kwinana; 3 years later the first tanker had come in. That was the span of the jo[illegible] The early preliminary planning had been in the first half of 1952; the Consulting Enginee[illegible] and contractors had been brought in on planning in the remainder of 1952. During 19[illegible] there had taken place all the really hard work of preparing the working drawings—

ourse, design had started earlier in a preliminary way. The first working drawings had een produced at the beginning of 1953. The first pile had been driven in the middle of 953. The construction had taken 18 months.

Mr Saurin suggested that that was worth bearing in mind—one whole year's planning, 6 months' lead of drawings over construction, 18 months to do the job.

Mr K. A. Spencer (Spencer & Partners) referred to the question of bonding between ıe steel piles. As could be seen in Fig. 25, it had been so designed as to prevent collecting ıe seaweed that drifted along the coastline, but the problem had been much more difficult ıan might appear. Probably had it been known a little earlier that cathodic protection 'as to be applied, a bar could have been provided for inserting through the concrete. He id not know whether that would have been possible from the civil engineering point of iew, but it would have greatly facilitated the eventual application of cathodic protection.

Three 150A transformer-rectifiers supplied direct current for the cathodic protection of ıe jetty. The positive leads ran out in the foreshore to three graphite ground beds. 'he current flowed out from the ground beds through the sea to any bare metal on the iles on both the jetty and dolphins, and returned through the bonding bar shown in 'ig. 25 back to the shore. The negative of the transformer-rectifiers was connected to ıe start of the jetty approach section.

Mr Spencer showed slides of the circuit involved and of the transformer-rectifiers in the athodic protection house. The operating voltage of the transformer-rectifiers was between 5 and 30 V, so the circuit resistance was very low.

The amount of current required in such a case depended entirely on the coating on the iles. It was important that a good coating should be applied. It had been found referable to have a hot-applied enamel coating. The bitumen types had been quite uccessful, although not always; it depended very much on the workmanship, which had een good at Kwinana. If workmanship was of high quality a very good current spread ould be obtained. Only by use of an electrically insulating coating and also cathodic rotection could corrosion of steel piles be prevented on a permanent basis of economic ost.

Mr A. H. Beckett (Chief Engineer, Sir Bruce White, Wolfe Barry & Partners) observed ıat there was no doubt in his mind that the engineers engaged in the work described new a lot about making concrete. Some of the crushing strengths they had obtained or the joint mortar appeared to be almost unbelievably high, and he would be interested o know what the Authors considered as the working stress for concrete that gave a cube trength of 10,000 lb/sq. in. after 7 days.

The ingenious methods adopted for making the joints between the precast concrete nits should go a long way towards providing the structural sturdiness and reserve trength that one associated with the more conventional type of cast-in-situ work.

In view of the excellence of the materials available for making concrete, would the uthors explain why, for piling, they had chosen steel box sections which had had to be ent out from Britain?

On almost every new oil jetty, one heard of another approach to the vexed problem of ow to restrain gently a charging oil tanker. There was no doubt that any form of esilience was a great advantage both to the jetty and the ship. The introduction of such esilience on a rigid jetty almost always involved the use of some form of mechanism, which worked a little less well if it was rusted up. In the work described in the Paper here was an excellent attempt to provide a simple and robust device which, taking dvantage of the small range of tide, ensured a centralized thrust on each rubber buffer y means of ball joints. Was there incorporated in that fender unit an hydraulic dash-pot ystem, and did the level of the sea modify the action of such a dash-pot. Was there the ossibility of an incompressible hydraulic cushion finding its way into the mechanism?

It was stated that cathodic protection was installed to prevent the corrosion of the steel- ork and the fender units. Could the Authors state if the intermittent dose of cathodic

protection that the fendering system received at high water was effective in retaining the working clearances of the sliding pistons?

Lastly, it was stated in the Paper that the fender system was designed to absorb the energy of a 40,000-ton vessel approaching at a speed of 1 ft/sec. The Paper also gave figures for energy absorption of the fender units, from which it was easy to calculate that twelve units must come into full working contact with the approaching vessel before the energy absorption could be found. Surely for that to happen the ship must lie truly parallel to the jetty face—a condition which would rarely obtain. The pilot would doubtless discover the limitations of that fendering system if he brought his ship at an angle to the jetty, but the price of his experience might be damage to the ship.

Mr Harry Ridehalgh (Sir William Halcrow and Partners, Consulting Engineers) said that the Paper was the third or fourth to be presented in recent years dealing with the provision of oil-loading facilities. It seemed now to be a regular feature of such Papers that whatever was described was bigger than anything yet built or had cost an astronomical amount. Curiously enough, the Authors had not mentioned the cost of the work, nor had anybody asked about it. The Authors had said, however, that it had been completed in 18 months and that the average labour force had been about 340—on a basic wage, plus overtime, which had probably come to about £17 a week. A little mental exercise showed that the labour bill on the job had been about £500,000. It was fairly easy from that to work up to quite a big figure for the cost of the work. He did not know whether the Authors would like to say how much it had cost, but one might well get up into the millions.

The Authors had given a good description of the plant used. There had been six tugs and launches, sixteen rowing boats, eighteen pontoons and barges, twelve derricks and cranes, five concrete mixers, two or three piling outfits, and odds and ends of heavy equipment in addition. That was quite formidable, but the Authors had agreed that it was fairly generous. The total weight of the piling had been about 4,400 tons. There had apparently been four or five senior engineers on the job and probably an equal number of junior engineers and inspectors. One could therefore get some idea of why the work had cost so much.

The point Mr Ridehalgh wanted to put was whether it had been unnecessarily expensive. The only comparison he had to offer was the three oil berths which his firm had built at Singapore, which were almost identical. They had been approached by piled spans, as at Kwinana, and there had been jetty heads. They had been designed to take 45,000-ton tankers as against the 32,000-ton tankers at Kwinana. The job had taken 16 months to build, it had cost £300,000, and the average labour force had been 150, as against 350 at Kwinana. The plant used had been two cranes, two piling rigs, two pontoons, two mixers, and one tug; whilst the total weight of the piling had been only about a quarter of that used at Kwinana. There had apparently been some meanness about the number of supervisory staff, since there had been only a resident engineer and one inspector; the contractor had had an agent and a junior engineer.

He would be interested to hear whether the Authors could offer any reasons for the very wide differences between the costs, plant, and labour on the two jobs.

Turning to the design of the jetty, Mr Ridehalgh hazarded the opinion that the roadway and trunkway design seemed unnecessarily complicated, and he was not very happy about using the particular type of prestressing in such close proximity to water. He felt that the jetty head was unnecessarily large having regard to the fact that only the centre 60 ft were used for oil handling facilities. At Singapore there had been a deck area of less than half that size, fronted by the berthing beam, 240 ft long, which absorbed the whole impact of the ships coming alongside. It might be that the size of the jetty had been dictated by the capacity to absorb ship blows, since Mr Cuerel thought it wise to have a large safety margin for the fenders. In the Singapore design the actual jetty head took no ship blow at all; it was all taken on the berthing beam—which brought him to the problem of energy absorbing fenders. Again, he thought that the design used was too

omplicated for practical use. Special castings were required; those might get broken r rust up and they had a relatively low energy absorbing capacity. Each block would, ı the travel given in the Paper, absorb rather less than 50 ft-tons. The energy to be bsorbed from a 32,000-ton tanker moving at 1 ft/sec was about twelve times that. Thus, or anything other than broadside blows, the eight fender blocks provided were going to e very much over-taxed or the jetty head would have to take a substantial portion of he blow. There appeared also to be no provision for the units to take longitudinal lows, although he knew that there was the stop at the end, but really that was only a tructural transfer of the blow to the rest of the jetty head. In addition, at least 60% f the jetty face was unprotected from either the normal berthing blow or the inevitable xceptional blow which usually put the berth out of commission. At Singapore the erthing beam absorbed at least twice as much energy (about 750 ft-tons) as all the eight nits would do at Kwinana, and the whole of the berth was protected even from the xceptional blow. That was absolutely essential, for the cost of a berth being out of ommission was exceptionally high, having regard to charter and demurrage rates at the resent time.

It appeared nowadays that they were tending to lose sight of the costs of jobs because hey thought they were perhaps getting something bigger and better and quicker than nything which had been built before.

Mr H. C. Visvesvaraya (Civil Engineering Department, Imperial College of Science nd Technology), referring to the jointing of the prestressed precast units and the use f a special stressing device, said that at the same time as the Kwinana project was in rogress stressing devices were being tried at the Imperial College with a view to developing device capable of stressing tendons from within small pockets or recesses in the members ı a convenient and safe manner. During that period a new jack was developed which as eminently suitable for establishing continuity between precast members. The jack ıat was made first was of a capacity of 25 tons and weighed less than 10 lb. Using the ıme principles a design for a jack with a capacity of 50 tons had been worked out; such jack would weigh only 18 to 20 lb. and would have an external diameter of 5½ in. compared with 82 lb. and 6½ in. respectively for the jack used on the Kwinana jetty, and 140 lb. nd 7½ in. respectively for the standard Lee-McCall jack of similar capacity. The small ze of the jack automatically resulted in much smaller pockets than those used in the winana project.

On p. 789 the Authors stated that the strength of the concrete cubes was 10,000 lb/sq. in. t 7 days, whereas on p. 807 they said that the works tests showed an average compressive rength of 10,000 lb/sq. in. at 28 days. Would the Authors please explain that apparent ıomaly?

He would also like to know whether the jointing faces were covered with neat cement rout before caulking.

*** **Mr F. P. Dath** (Civil Engineer, Central Electricity Authority) thought it seemed little unfair and pointless to try to make a comparison between the costs of the Singapore ıd the Kwinana jetties, unless the labour costs which appeared to be the salient issue ere considered in their relative proportion, irrespective of the design aspects. It was ell known that the basic wage for a general labourer in Singapore was about 1/10th of ıat mentioned on pp. 812 and 820; therefore, since labour costs formed, in most cases, the ajor part of the cost in civil engineering works, that aspect alone was sufficient to rule ıt a direct comparison between the two constructions.

He had not carried out any construction works in Australia, but he had been acquainted ith some of the difficulties in the labour problems there, and he thought it was generally cognized that Australian labour was the most expensive in the world. He was, of urse, referring to the Australian and not to the European immigrant.

*** This contribution was submitted in writing after the closure of the oral discussion. -SEC.

Mr Murray, in reply, referred to the 15-ton limit for the weight of the precast units mentioned by Mr Cuerel and others. The decision to limit the loads to 15 tons had been made after long and serious consideration by all parties in the light of known and antici-pated site conditions and the availability and cost of plant. The works had been completed in excellent time, and therefore, it could be said that the decision was sound. At the same time, however, he thought that given suitable conditions and circumstances, great econ-omies in construction could be made on similar works by precasting on a much heavier scale.

Mr Ridehalgh had criticized the cost of the works and had given some figures of a job in Singapore as a comparison. Mr Murray thought it was futile to compare one job with another without having knowledge of the respective requirements and conditions. Mr Ridehalgh had given only one clue—the quantity of piling at Kwinana was four times tha at Singapore. If one followed that clue, it seemed clear that the essential work a Kwinana was several times greater than at Singapore. Yet the average labour force wa 150 at Singapore; 340 at Kwinana. The time for completion at Singapore was 16 months at Kwinana, allowing for preliminary works, 20 months. Mr Murray understood that th total cost of the Kwinana Jetties was about £1,700,000. When account had been taken of the scale of the work, the respective site conditions, and of the tremendous difference i labour rates, he thought that in the matter of cost comparison the boot might well be o the other foot.

Mr Collett, in reply, said that the particular criticisms of the fendering arrangement mentioned by Mr Braddick had not in fact been borne out in practice. There was n evidence to suggest that the fenders were being driven fully home and the fixed fenderin above the flexible fender units was virtually untouched.

Mr Spencer had made reference to the bonding between the steel piles, and Mr Collet agreed that had the details of the cathodic protection been finalized at an earlier stage o the jetty design it might have been possible to cast bonding bars into the concrete unit for that purpose. However, the method adopted was both neat and practicable an equally facilitated the eventual application of the cathodic protection.

In reply to Mr Beckett's queries regarding the working stress in the concrete used in th caulked joints, Mr Collett explained that the main object had been to obtain a fine dr concrete which, upon being rammed hard in the joint space, could then be immediatel stressed. The cube strengths of 10,000 lb/sq. in. at 7 days thus obtained, had bee pleasing, but such high strengths were not regarded as essential, bearing in mind th comparatively low working stress in the joint concrete.

With reference to Mr Beckett's remarks about the fender units, Mr Collett went on say that the water which found its way inside the buffers did in fact escape freely whe the fender units were compressed and there was no possibility of an incompressib hydraulic cushion forming. Regarding the efficiency of the cathodic protection and th retention of working clearances on the sliding pistons, Mr Collett said that such clearanc were being retained, and after approximately 18 months in operation there was no sig of corrosion of the units.

In reply to Mr Beckett's remarks that twelve units must come into full working conta with a 40,000-ton tanker approaching at 1 ft/sec to bring it to a halt, Mr Collett said th that was incorrect since energy was absorbed in other ways than by the fenders themselv In fact, it was a common assumption that only about half of the total energy was tran ferred to the fender system. In his estimation four to five units would safely halt tanker berthing at the specified speed and that was proving to be the case.

With regard to Mr Ridehalgh's contribution, Mr Collett mentioned that he wou confine his remarks to the design problems only. Mr Ridehalgh had remarked that would not have used the particular type of prestressing in such close proximity to wa but had not gone on to say why he would not have specified it. Mr Collett could not why the system adopted was not the equal of any other system of prestressing for use in marine structure; the steel was adequately protected by a fine dense concrete against t effect of sea-water and salt-laden atmosphere. Mr Ridehalgh had suggested that t

nder design was too complicated for practical use and yet, in practice, the fenders were rforming a good job of work. Mr Ridehalgh had also raised the same point as Mr Beckett garding the energy absorption of the fender system and the previous answer applied. r Ridehalgh had further suggested that the jetty heads were unnecessarily large (appantly at Singapore they were very much smaller). Mr Collett did not have sufficient nowledge of the details of the Singapore jetty to make a comparison, but the jetty heads Kwinana with the number of pipes required, hose-handling equipment, fire-fighting stallations, mooring arrangements, office, etc., had proved not to be unnecessarily large.

Mr Visvesvaraya had referred to a new jack which had been developed at the Imperial ollege and which appeared to offer certain advantages over the type of jack used at winana. Mr Collett mentioned that when it was first suggested to Messrs McCall that eir standard jack could be reduced in size and weight they felt it would be extremely fficult. However, they were able to bring it down from 140 lb. and 7¼ in. dia. to 82 lb. d 6½ in. dia. Mr Collett would be pleased to see an equivalent jack with a weight of 3 to 20 lb. and he was sure that it would be welcomed generally.

Mr Visvesvaraya had drawn attention to an apparent anomaly in the concrete cuberength figures mentioned on pp. 789 and 807. The figures on p. 789 related to the cubes ade at Feltham during the trials, while those on p. 807, which are somewhat lower, d been obtained under site conditions.

Correspondence on this Paper is now closed.—SEC.

Paper No. 6116

STRESS ANALYSIS OF THREE-PINNED ARCH-RIBBED DOMES

by
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and
Madhukar Narayan Gogate, B.E., M.Sc.

(*Ordered by the Council to be published with written discussion*)

SYNOPSIS

Braced and ribbed domes are often used for covering large areas. The Paper discusses the principles of the stress analysis of three-pinned ribbed domes consisting of any number of three-hinged semi-circular arches, interconnected together at the apex. General formulae of reaction components are developed, facilitating the determination of influence lines of these functions. A model of a dome having eight ribs was used for the verification of the derived formulae and showed a very satisfactory agreement between the theory and the experiment.

Introduction

Space structures, of which domes are typical examples, are generally used wher large areas have to be covered. They possess several distinct advantages wher compared with the conventional two-dimensional roof trusses. They have grea stiffness, are of good appearance, and owing to the lack of internal columns o horizontal bracing members have a completely unobstructed inner space. A goo indication of the performance of any building frame is the structural weight require to cover a square foot of the floor area. Braced and ribbed domes compare ver favourably in this respect with any other form of construction and are becomin popular.[1]

The Paper describes the principles of the stress analysis of one of the most popula types of ribbed domes—the three-pinned arch-ribbed construction. The principa types of ribbed domes, used for covering exhibition halls, stadiums, planetarium etc., are shown in Fig. 1. In general, ribbed domes consist of a number of radia solid or trussed (latticed) ribs, interconnected at the crown and supported in a adequate way at the foundation. If the ribs are directly pin-connected to founda tion blocks, the dome is of the unstiffened type. If, however, the ribs are connecte at the bottom to an elastic base ring, the dome is of the stiffened type, and the rea tion components from the ribs on the ground are only vertical, the thrust being take by the base ring.[2] In practice the rigidity of the connexion of all the ribs at th apex depends upon the method of construction. If the connexions are welded th apex joint is fully rigid, introducing fixing moments, which makes the stress analys

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[1] The references are given on p. 844.

such a dome very tedious.[3–7] The fixity of all ribs at the crown, though having definite influence on the stress distribution of small span domes, can be neglected r larger spans, especially for the steel ribbed type, having slender ribs. It is often npossible and nearly always uneconomical to provide footings which would cometely fix the ribs against rotation and it is generally assumed that ribbed domes e pinned at their foundation. It should be noted that a ribbed dome, if fixed

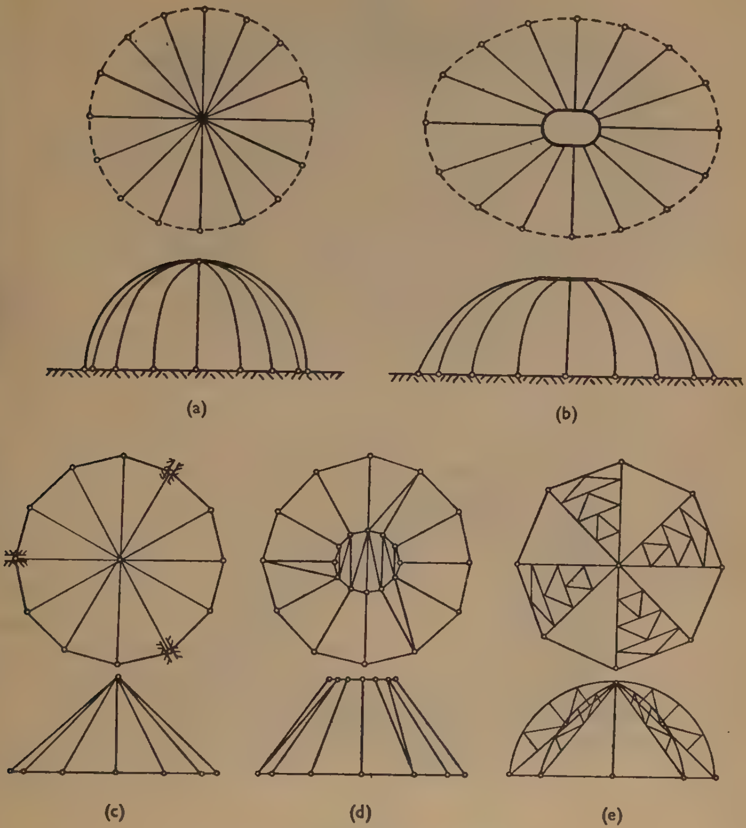
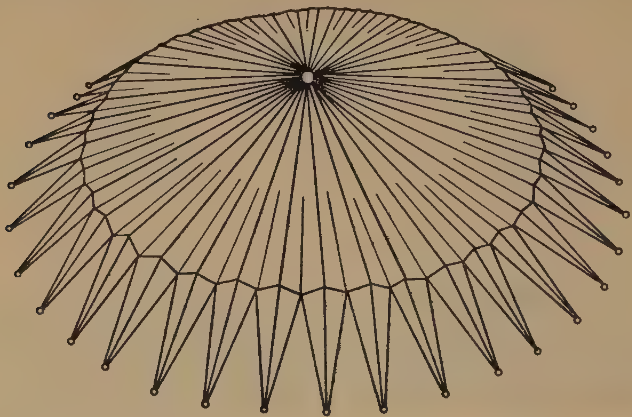


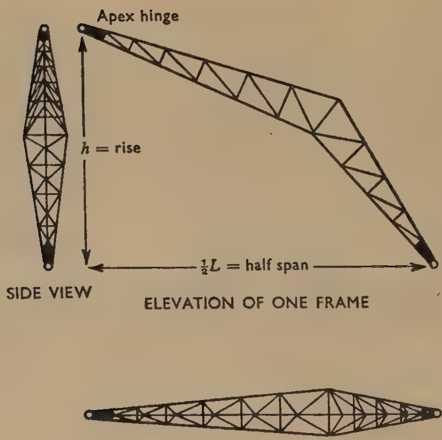
FIG. 1

dly at the apex, can resist a perfectly general system of loading, whereas if the d joint at the apex is replaced by a spherical hinge the resulting three-pinned ed dome will be able to resist only loads acting in the vertical planes passing ough the neutral axis of the ribs. Such a vertical system is of primary imtance in practical design. Lateral forces are always assumed to be taken by the covering or resisted by the horizontal purlins, considered as simply supported he ribs. In practice three-pinned domes also often have diagonal members, necting two ribs together, as in Fig. 1e. Fig. 1b shows a ribbed dome covering lliptical area, with a lantern ring at the top. If the diameter of this ring is small

compared with other dimensions of the structure, it is usual to assume in the analysis that all ribs intersect at the common joint. Such ribbed domes have a high degree of redundancy and this is the only justification for the very approximate analyses often used for their design.



(a)



(b)

FIG. 2

In a general case the degree of redundancy of a dome consisting of n arches ful encastré at the supports and rigidly interconnected at the apex is

$$N = 6(2n - 1) \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (1)$$

, if all the supports are hinged:

$$N = 3(2n - 2) \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (2)$$

ı a general case, for a three-pinned ribbed dome, having a hinged apex joint and ıder the action of any system of loads acting in the vertical planes of the ribs, ıe degree of redundancy becomes:

$$N = 2n - 3 \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (3)$$

ı practice the number of ribs is large and, therefore, the stress analysis of a three-nned dome by the classical approach would become very complicated. However, ıe analysis is simplified if all the $2n - 3$ unknown components are expressed in :rms of the deflexion of the apex joint 0, which is the same for all ribs.

For small spans, straight members are used and form the tent roofs (Figs 1c and 1), which are sometimes confused with the true ribbed domes. The main difference etween tent roofs and ribbed domes is that the straight members of the former are ıeoretically under the action of axial forces only, whereas in ribbed domes the ıembers, even if pin-connected, are stressed by bending moments, shearing forces, ıd axial loads. Tent roofs are popular for small towers and can be made statically eterminate by an appropriate arrangement of the supports.

For larger spans the arrangement shown in Fig. 1d is frequently used. This ructure can be statically determinate and stable for any system of loading if all ıe supports have a freedom of movement in a predetermined direction, i.e., if there :e only two reaction components at each support. It should be noted, however, ıat several panels forming the inclined sides of the dome must be stiffened by iagonal members, able to resist compression as well as tension. The structure ould otherwise become unstable and collapse owing to the rotation of the stiff top ınel under lateral loads. This instability is identical to that which may occur ith plane spoked wheels. In such wheels the wire spokes cannot be made radial, r they are unable to transmit tangential loads from the rim, without a rotation of ıe rim relative to the hub. The difficulty in this case can be overcome by inclining ıe spokes from the radial direction.[8]

Three-pinned ribbed domes are often used in modern construction and consist sually of prefabricated units (ribs), which can be speedily erected on the site with ttle scaffolding. Fig. 2a is a diagrammatic representation of a three-pinned dome sed for covering a large area. All the ribs are prefabricated steel units, of triangular ross-section as in Fig. 2b.

Analysis

:) *Notation*

Three-pinned ribbed domes encountered in practice are in almost all cases cyclically ymmetrical, having ribs of semi-circular shape and of constant moment of inertia I. he analysis which follows is carried out for such a typical dome. This analysis will e also directly useful for other types of ribbed domes, having for instance rigid onnexions, since in such cases the "three-pinned" dome is generally taken as the rimary structure, on which the effects of the rigidity of connexions are imposed eparately.[9] The notation used is shown in Fig. 3; 1—0, 2—0, . . . r—0, . . . n—0, —0, 2′—0, . . . r'—0, . . . n'—0 are $2n$ ribs forming the dome, numbered in a ockwise direction, n may be odd or even. Any load P acting on the structure can e resolved into horizontal and vertical components P_h and P_v. The point of pplication of the load is defined by the angle α. Because of the symmetry of the

structure the effect of unit loads acting on one rib, say, rib 1—0, need only be studied This rib 1—0 will be called the directly loaded rib. The position of any rib r—0 not directly loaded, is described by the angle θ_r, measured in a clockwise directio from the base line 1—1′. More complicated loading systems will be dealt wit using the principle of superposition. Downward vertical and inward horizonta forces are considered as positive; the deflexion of the apex joint 0 is positive, i

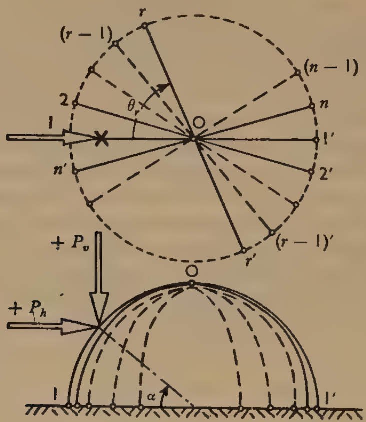


Fig. 3

following the positive directions for forces acting on the rib 1—0. Two cases o loading will be considered separately:—

Case I: a unit horizontal load in plane of rib 1—0.
Case II: a unit vertical load in plane of rib 1—0.

Each case will be resolved into symmetrical and skew-symmetrical loading. (Case IA and IB, IIA and IIB.)

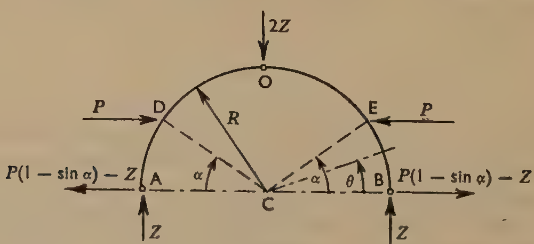


Fig. 4

The dome consists of n three-pinned arches r—0—r', interconnected by th common apex hinge. General formulae will first be derived for the vertical an horizontal displacements $\delta_v{}^0$ and $\delta_h{}^0$ of the apex of a three-pinned arch, under uni vertical and horizontal loads.

General expressions of the apex displacements

Under horizontal symmetrical loads P (Case IA) acting on the three-pinned arch, e apex will deflect only vertically (see Fig. 4). This vertical displacement will be tained by the strain energy method using the first theorem of Castigliano.

Assuming a fictitious vertical force $2Z$ at 0:

$$\frac{\partial U}{\partial(2Z)_{Z=0}} = \delta_v{}^0$$

x is any point on the rib, at an angular distance θ from CB, the bending moment x is:

r the part BE

$$M = ZR(1 - \cos\theta) + \{P(1 - \sin\alpha) - Z\}R\sin\theta \quad . \quad . \quad (4a)$$

$$\frac{\partial M}{R\partial Z} = 1 - \cos\theta - \sin\theta$$

erefore

$$\frac{\partial U}{\partial(2Z)_{Z=0}} = \frac{PR^3}{2EI}(1 - \sin\alpha)\left(1 - \cos\alpha - \frac{\sin^2\alpha}{2} - \frac{\alpha}{2} + \frac{\sin\alpha\cos\alpha}{2}\right) \quad . \quad . \quad (4b)$$

r the part EO

$$M = ZR(1 - \cos\theta) + \{P(1 - \sin\alpha) - Z\}R\sin\theta - PR(\sin\theta - \sin\alpha) \quad (5a)$$

$$\frac{\partial M}{R\partial Z} = 1 - \cos\theta - \sin\theta$$

erefore

$$\frac{\partial U}{(2Z)_{Z=0}} = \frac{PR^3}{2EI}(1 - \sin\alpha)$$

$$\left(\frac{3\pi}{4} - \frac{1}{2} - \frac{3\alpha}{2} - 2\cos\alpha + \sin\alpha - \frac{\sin^2\alpha}{2} + \frac{\sin\alpha\cos\alpha}{2}\right) \quad . \quad . \quad (5b)$$

d

$$\sum\frac{\partial U}{\partial(2Z)_{Z=0}} = \delta_v{}^0 = \frac{PR^3}{EI}$$

$$\left(1 - \frac{\alpha}{2} - \alpha\sin\alpha + \frac{3\pi - 6}{4}\sin\alpha - \cos\alpha - \frac{1}{2}\sin\alpha\cos\alpha + \frac{1}{2}\sin^2\alpha\right) \quad . \quad (6)$$

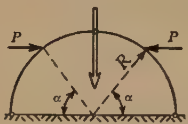
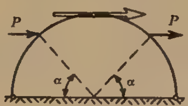
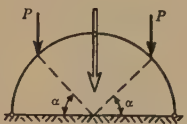
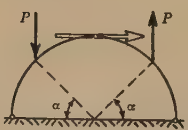
imilar expressions have been obtained for other systems of loading. It is seen that e vertical and horizontal displacements of the apex can be expressed for any stem of loading by the following general formula:

$$= \frac{PR^3}{EI}(a + b\alpha + c\alpha\sin\alpha + d\alpha\cos\alpha + e\sin\alpha +$$

$$f\cos\alpha + g\sin\alpha\cos\alpha + h\sin^2\alpha + i\cos^2\alpha) \quad . \quad . \quad . \quad . \quad . \quad (7)$$

he numerical values of the coefficients $a, b, c, \ldots i$, are given for each of the four ading cases IA, IB, IIA, IIB in Fig. 5.

The displacement in each case is a function of the angle α which denotes the point

Case	Loading system	1	α	$\alpha\sin\alpha$	$\alpha\cos\alpha$	$\sin\alpha$	$\cos\alpha$	$\sin\alpha\cos\alpha$	$\sin^2\alpha$	$\cos^2\alpha$
		multiply the above terms by the coefficients								
		a	*b*	*c*	*d*	*e*	*f*	*g*	*h*	*i*
IA	$\delta_v^{I} = \frac{PR^3}{EI}\cdot f_1(\alpha)$ 	1	$-\frac{1}{2}$	-1	0	$\frac{3\pi-6}{4}$	-1	$-\frac{1}{2}$	$+\frac{1}{2}$	0
IB	$\delta_h^{II} = \frac{PR^3}{EI}\cdot f_2(\alpha)$ 	-1	$+\frac{1}{2}$	$+1$	0	$\frac{\pi-6}{4}$	$+1$	$+\frac{1}{2}$	$-\frac{1}{2}$	0
IIA	$\delta_v^{III} = \frac{PR^3}{EI}\cdot f_3(\alpha)$ 	$\frac{3\pi-8}{4}$	$+\frac{1}{2}$	0	$+1$	-1	$\frac{6-3\pi}{4}$	$-\frac{1}{2}$	0	$+\frac{1}{2}$
IIB	$\delta_h^{IV} = \frac{PR^3}{EI}\cdot f_4(\alpha)$ 	$\frac{\pi-4}{4}$	$-\frac{1}{2}$	0	-1	$+1$	$\frac{6-\pi}{4}$	$+\frac{1}{2}$	0	$-\frac{1}{2}$

The open arrow at the apex indicates the direction of the displacement

Fig. 5.—Deflexions of the apex of the three-pinned arch

f application of the load and it is convenient to use the following functions $f(\alpha)$ for ach loading system:

$$\left.\begin{aligned}
(\alpha) &= 1 - \frac{1}{2}\alpha - \alpha\sin\alpha + \frac{3\pi-6}{4}\sin\alpha \\
&\qquad - \cos\alpha - \frac{1}{2}\sin\alpha\cos\alpha + \frac{1}{2}\sin^2\alpha \\
(\alpha) &= -1 + \frac{1}{2}\alpha + \alpha\sin\alpha + \frac{\pi-6}{4}\sin\alpha \\
&\qquad + \cos\alpha + \frac{1}{2}\sin\alpha\cos\alpha - \frac{1}{2}\sin^2\alpha \\
(\alpha) &= \frac{3\pi-8}{4} + \frac{1}{2}\alpha + \alpha\cos\alpha - \sin\alpha \\
&\qquad + \frac{6-3\pi}{4}\cos\alpha - \frac{1}{2}\sin\alpha\cos\alpha + \frac{1}{2}\cos^2\alpha \\
(\alpha) &= \frac{\pi-4}{4} - \frac{1}{2}\alpha - \alpha\cos\alpha + \sin\alpha \\
&\qquad + \frac{6-\pi}{4}\cos\alpha + \frac{1}{2}\sin\alpha\cos\alpha - \frac{1}{2}\cos^2\alpha
\end{aligned}\right\} \quad . \quad . \quad . \quad (8)$$

s a check it should be observed that:

$$f_1(\alpha) + f_4(90° - \alpha) = 0$$
$$f_2(\alpha) + f_3(90° - \alpha) = \pi - 3 = 0{\cdot}1416$$

able 1 gives the numerical values of $f(\alpha)$. These functions are also represented aphically in Fig. 6, which may be used to determine these values for any value α.

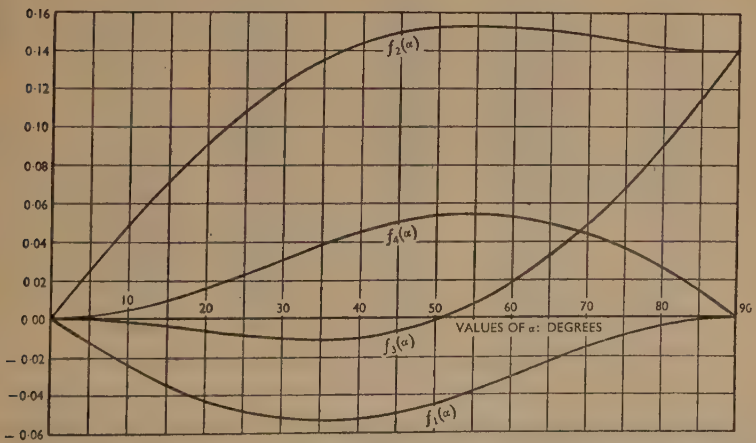


Fig. 6

General expressions of the reaction components at any support of the dome

ase IA.—Symmetrical horizontal loads acting on the arch 1—0—1′ (Fig. 7). the ribs, forming the dome, are interconnected at the common hinge 0 and all

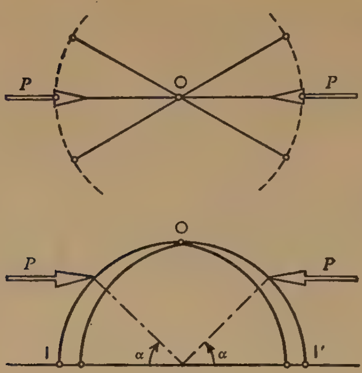


FIG. 7

undergo the same vertical displacement $\delta_v{}^0$ of the apex joint. Owing to this inter connexion, the external loading applied to the arch 1—0—1′ produces equal vertica reactions $2Y_1$ acting at the apex on all the other arches not directly loaded. Fron

TABLE 1

α	$f_1(\alpha)$	$f_2(\alpha)$	$f_3(\alpha)$	$f_4(\alpha)$
0°	0·0000	0·0000	0·0000	0·0000
5°	− 0·0125	+ 0·0248	− 0·0006	+ 0·0012
10°	− 0·0242	+ 0·0487	− 0·0020	+ 0·0042
15°	− 0·0344	+ 0·0712	− 0·0044	+ 0·0092
20°	− 0·0431	+ 0·0915	− 0·0071	+ 0·0156
25°	− 0·0493	+ 0·1091	− 0·0094	+ 0·0227
30°	− 0·0530	+ 0·1238	− 0·0115	+ 0·0305
35°	− 0·0547	+ 0·1358	− 0·0125	+ 0·0381
40°	− 0·0531	+ 0·1441	− 0·0114	+ 0·0445
45°	− 0·0498	+ 0·1498	− 0·0082	+ 0·0498
50°	− 0·0445	+ 0·1530	− 0·0025	+ 0·0531
55°	− 0·0381	+ 0·1541	+ 0·0058	+ 0·0547
60°	− 0·0305	+ 0·1531	+ 0·0178	+ 0·0530
65°	− 0·0227	+ 0·1510	+ 0·0325	+ 0·0493
70°	− 0·0156	+ 0·1489	+ 0·0501	+ 0·0431
75°	− 0·0092	+ 0·1460	+ 0·0704	+ 0·0344
80°	− 0·0042	+ 0·1436	+ 0·0929	+ 0·0242
85°	− 0·0012	+ 0·1422	+ 0·1168	+ 0·0125
90°	0·0000	+ 0·1416	+ 0·1416	0·0000

Table 1 it may be seen that the vertical displacement of the crown point of any arc r—0—r' caused by a unit vertical load at 0 equals:

$$\frac{1}{2}\cdot\frac{R^3}{EI}\cdot f_3(90°) = \frac{1}{2}\cdot\frac{R^3}{EI}\times 0{\cdot}1416 = 0{\cdot}0708\frac{R^3}{EI} \quad . \quad . \quad . \quad (9)$$

The directly loaded arch 1—0—1′ is under the action of two horizontal forces P an an upward force $2Y_1(n-1)$ representing the reaction from the remaining $(n-$

ches. This may be seen by reference to Fig. 9 (p. 836), Case IA. Under these rces the apex of the considered arch moves vertically by:

$$\frac{PR^3}{EI}f_1(\alpha) - (n-1)2Y_1\frac{0{\cdot}0708R^3}{EI} \quad . \quad . \quad . \quad . \quad (10)$$

ving to the interconnexion, vertical displacements of the apex of the directly aded arch 1—0—1′ and any arch r—0—r' must be equal or

$$0{\cdot}0708\frac{R^3}{EI} \times 2Y_1 = \frac{PR^3}{EI} \cdot f_1(\alpha) - (n-1) \,.\, 2Y_1\frac{0{\cdot}0708R^3}{EI} \quad . \quad (11)$$

om which it follows that

$$Y_1 = \frac{P}{0{\cdot}1416n} \cdot f_1(\alpha) \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (12)$$

d the vertical displacement of the apex hinge in the dome:

$$\delta_v{}^0 = \frac{PR^3}{nEI} \cdot f_1(\alpha) \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (13)$$

Case IB.—Skew-symmetrical horizontal loads P acting on the arch 1—0—1′ ig. 8). Under this loading the apex of the dome moves horizontally in the plane the loaded arch 1—0—1′ by δ. The effect of the interconnexion is that any arch- r—0—r', not directly loaded, has its apex horizontally displaced in its own plane

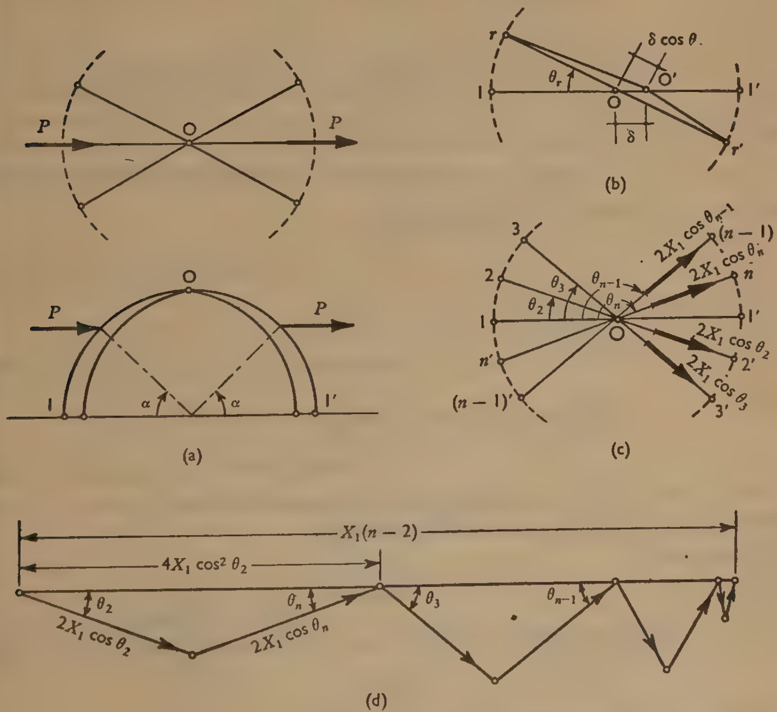


FIG. 8

by $\delta \cos \theta_r$ as in Fig. 8b. It should be noted that the displacement δ is very smal compared with other dimensions of the structure and does not change the geometr of the dome, so that the value of the angle θ_r remains unaffected. Any arch r—0— whose apex moves by $\delta \cos \theta_r$, is therefore pulled by a certain force, proportional t $\cos \theta_r$. This force is horizontal, acts in the plane of the arch, and is denoted in th subsequent analysis by $2X_1 \cos \theta_r$.

By symmetry: $\theta_2 + \theta_n = \theta_3 + \theta_{n-1} = \ldots \quad = \theta_r + \theta_{n-r+2} = \pi$

also $\theta_2 = \frac{\pi}{n}, \theta_3 = \frac{2\pi}{n} \ldots$ or, generally, $\theta_r = \frac{(r-1)\pi}{n}$

Considering therefore two symmetrical arches, say, 2—0—2′ and n—0—n', whic are under the action of $2X_1 \cos \theta_2$ and $2X_1 \cos \theta_n$ respectively, it is clear that th resultant of these two forces is equal to a horizontal force $4X_1 \cos^2 \theta_2$, acting alon the direction 1—0—1′ as in Fig. 8d. Such a resultant can be found for any pair c symmetrical arches and therefore the total horizontal force, representing the result ant of all these forces $2X_1 \cos \theta_r$ is given by the following equation:

$$\sum_{r=2}^{r=\frac{n}{2}} 4X_1 \cos^2 \theta_r = 4X_1[\Sigma \cos^2 \theta_2 + \cos^2 \theta_3 + \ldots + \cos^2 \theta_{n/2}] \quad . \quad . \quad . \quad (14)$$

$$= 4X_1\left[\cos^2 \frac{\pi}{n} + \cos^2 \frac{2\pi}{n} + \ldots + \cos^2 \left(\frac{n}{2} - 1\right)\frac{\pi}{n}\right] \text{ for an eve}$$

value of n

and $$= 4X_1\left[\cos^2 \frac{\pi}{n} + \cos^2 \frac{2\pi}{n} + \ldots + \cos^2 \left(\frac{n-1}{2}\right)\frac{\pi}{n}\right] \text{ for an od}$$

value of n.

Putting $\beta = \frac{2\pi}{n}$ and using the relation $2 \cos^2 \theta = 1 + \cos 2\theta$ the resultant horizonta force given by equation (14) is expressed as:

$$\left.\begin{aligned} \sum_{r=2}^{r=\frac{n}{2}} 4X_1 \cos^2 \theta_r &= 2X_1\left[\frac{n-2}{2} + \cos\beta + \cos 2\beta + \ldots + \cos \frac{n-2}{2}\beta\right] \\ &\qquad \text{if } n \text{ is even} \\ \text{or} \quad &= 2X_1\left[\frac{n-1}{2} + \cos\beta + \cos 2\beta + \ldots + \cos \frac{n-1}{2}\beta\right] \\ &\qquad \text{if } n \text{ is odd} \end{aligned}\right\} \quad . \quad (15)$$

It will be proved now that equation (15) can be expressed by one very simple formula which is equally valid for even or odd values of n.

Observing that

$$\cos\phi + \cos 2\phi + \ldots + \cos n\phi = \frac{\cos\left(\frac{n+1}{2}\right)\phi \sin \frac{n}{2}\phi}{\sin \frac{\phi}{2}} \quad . \quad (16)$$

it will be possible to simplify equation (15) as follows.

If n is even: $\cos\beta + \cos 2\beta + \ldots + \cos\frac{n-2}{2}\beta$ reduces to 0; since $\beta = \frac{2\pi}{n}$,

$\frac{n\beta}{4} = \frac{\pi}{2}$.

If n is odd: $\cos\beta + \cos 2\beta + \ldots + \cos\frac{n-1}{2}\beta$ reduces to $\dfrac{\sin -\frac{\beta}{2}}{2\sin\frac{\beta}{2}} = -\frac{1}{2}$;

ince $n\beta/2 = \pi$ and $\sin\pi = 0$.

Adding these results to the corresponding constant terms contained in equation 16), for an even value of n:

$$2X_1\left[\frac{n-2}{2} + 0\right] = X_1(n-2)$$

nd for an odd value of n:

$$2X_1\left[\frac{n-1}{2} - \frac{1}{2}\right] = X_1(n-2)$$

t has been proved therefore that in each case, for n even or odd, the resultant orizontal force becomes:

$$\sum_{r=2}^{r=n/2} 4X_1\cos^2\theta_r = X_1(n-2) \quad \ldots\ldots \quad (17)$$

This force $X_1(n-2)$ representing the resultant of all forces $2X_1\cos\theta_r$ acting on all —0—r' arches is directed towards support 1′.

An equal force, but acting in the opposite direction, applies at 0 on the directly oaded arch 1—0—1′. A unit horizontal load at the apex of a three-pinned arch vould produce a horizontal deflexion equal to

$$\frac{1}{2} \times 0{\cdot}1416\frac{PR^3}{EI} = 0{\cdot}0708\frac{R^3}{EI}$$

Therefore, the deflexion of any arch r—0—r' is

$$\delta\cos\theta_r = 2X_1\cos\theta_r\left(0{\cdot}0708\frac{R^3}{EI}\right)$$

Similarly, by superposition the deflexion of the arch 1—0—1′ is

$$\delta = f_2(\alpha)\,.\,\frac{PR^3}{EI} - (n-2)X_1\left(0{\cdot}0708\frac{R^3}{EI}\right)$$

From these equations it follows that

$$\delta = 2X_1\,.\,\frac{0{\cdot}0708R^3}{EI}$$

nd

$$X_1 = \frac{P}{0{\cdot}0708n}\cdot f_2(\alpha) \quad \ldots\ldots\ldots\ldots \quad (18)$$

ntroducing now the known value of X_1 into the above expression of δ the final value f the displacement

$$\delta = \frac{PR^3}{EI}\cdot\frac{2}{n}\cdot f_2(\alpha) \quad \ldots\ldots\ldots \quad (19)$$

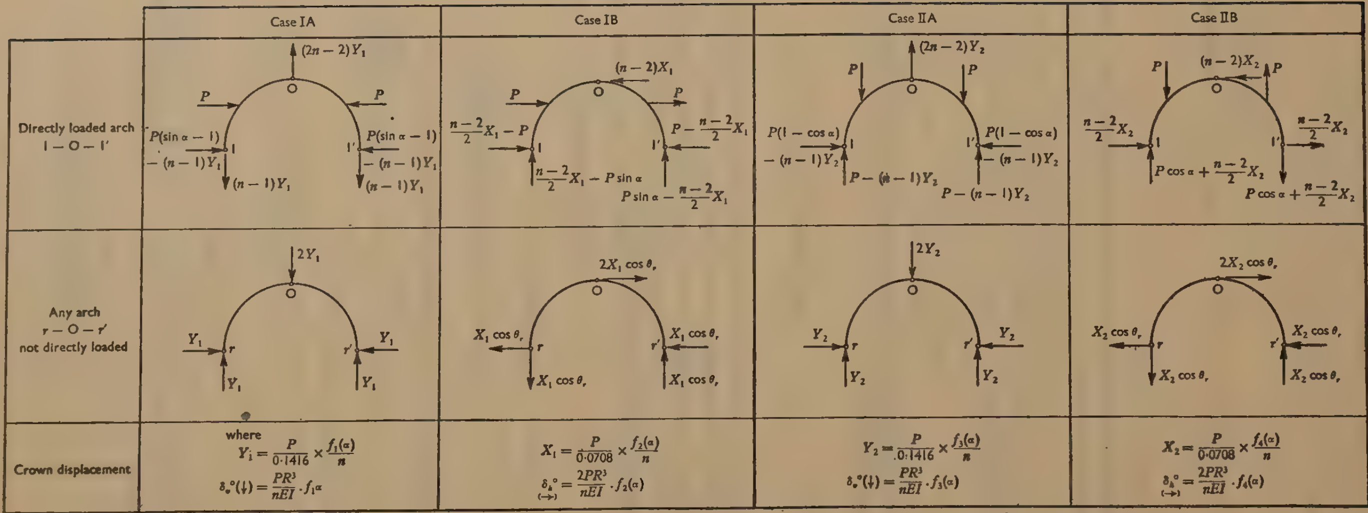


FIG. 9.—REACTIONS AND FORCES ACTING ON RIBS

The last two equations are valid for all values of $n \geqslant 2$. If $n = 2$, the arch 2—0—2′ will be unstressed under the considered system of loading.

Thus knowing all the forces acting on all the ribs, the general expressions for the reaction components for any support can be readily determined and are given in Fig. 9. By superposition, the results for Cases IA and IB added and divided by $2P$, give the reactions caused by a unit horizontal load. Results for vertical load (Case II) have been obtained in a very similar way. Fig. 10 gives the summary of the results, thus enabling a quick calculation of the reactions for any support of a dome consisting of any number of ribs. Knowing the reactions at any support of the dome, it is possible to determine in each particular case the bending moment, shearing force, or axial force at any point. It is also possible to derive general expressions for these functions for any system of loading. Such expressions will be found useful during design when the diagrams of bending moments or shearing forces are to be prepared.

Having the general formulae, valid for any number of ribs, influence lines of reaction components at any support, or influence lines of bending moments, shearing forces, and axial forces for any particular point in any of the ribs can be prepared without difficulty and very quickly. Such influence lines would be very useful when complicated loading systems are considered, consisting of concentrated or distributed loads; determination of maximum stresses owing to live load (wind forces) then becomes very simple.

Example.—To illustrate the simplicity of the discussed method, a three-pinned dome consisting of four arches will be considered. The structure is under the action of a horizontal force $P_h = 1$ ton, applied to the rib 1—0 at $\alpha = 45°$ (loading Case I). First the values of the $f(45°)$ will be found from Table 1.

$$f_1(\alpha) = f_1(45°) = -\,0{\cdot}0498$$
$$f_2(\alpha) = f_2(45°) = +\,0{\cdot}1498$$
$$f_3(\alpha) = f_3(45°) = -\,0{\cdot}0082$$
$$f_4(\alpha) = f_4(45°) = +\,0{\cdot}0498$$

n denotes the number of arches (= 4)
Therefore, $\theta_2 = 45°$; $\theta_3 = 90°$; $\theta_4 = 135°$
$\alpha = 45°$: $\sin\alpha = \cos\alpha = 0{\cdot}7071$

The reaction components at any support under this load can now be found quickly using the formulae given in Fig. 10.

Reaction components at the support 1.

(*a*) Vertical: $$V_1 = \frac{\sin\alpha}{2} + \frac{1}{0{\cdot}2832n}\{(n-1)f_1(\alpha) - (n-2)f_2(\alpha)\}$$

$$= 0{\cdot}3535 + \frac{1}{0{\cdot}2832 \times 4}\{-3 \times 0{\cdot}0498 - 2 \times 0{\cdot}1498\}$$
$$= -\,0{\cdot}0428 \text{ ton}$$

(*b*) Horizontal: $$H_1 = \frac{\sin\alpha}{2} - 1 + \frac{1}{0{\cdot}2832n}\{-(n-1)f_1(\alpha) + (n-2)f_2(\alpha)\}$$

$$= 0{\cdot}3535 - 1 + \frac{1}{0{\cdot}2832 \times 4}\{3 \times 0{\cdot}0498 + 2 \times 0{\cdot}1498\}$$
$$= -\,0{\cdot}2500 \text{ ton}$$

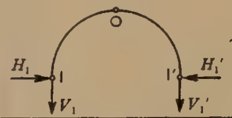
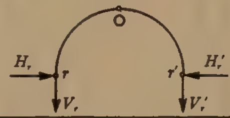
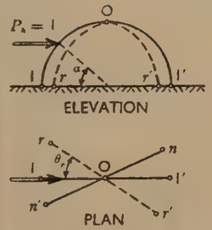
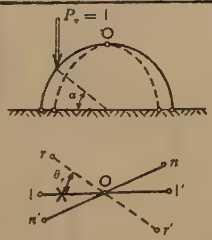
<table>
<tr><th>Case</th><th>Type of loading</th><th>Reactions for directly loaded arch 1—1′

</th><th>Reactions for any not directly loaded arch r — r′

</th><th>Deflexions of the apex O</th></tr>
<tr><td>I</td><td>
</td><td>Vertical reactions
$V_1 = \frac{\sin\alpha}{2} + \frac{1}{0\cdot2832n}\{(n-1)f_1(\alpha) - (n-2)f_2(\alpha)\}$
$V_1' = -\frac{\sin\alpha}{2} + \frac{1}{0\cdot2832n}\{(n-1)f_1(\alpha) + (n-2)f_2(\alpha)\} = -H_1'$
Horizontal reactions
$H_1 = \frac{\sin\alpha}{2} - 1 + \frac{1}{0\cdot2832n}\{-(n-1)f_1(\alpha) + (n-2)f_2(\alpha)\}$
$H_1' = \frac{\sin\alpha}{2} + \frac{1}{0\cdot2832n}\{-(n-1)f_1(\alpha) - (n-2)f_2(\alpha)\}$</td><td>Vertical reactions
$V_r = \frac{1}{0\cdot2832n}\{-f_1(\alpha) + 2f_2(\alpha)\,.\,\cos\theta_r\} = -H_r$
$V_1' = \frac{1}{0\cdot2832n}\{-f_1(\alpha) - 2f_2(\alpha)\,.\,\cos\theta_r\} = -H_r'$
Horizontal reactions
$H_r = \frac{1}{0\cdot2832n}\{+f_1(\alpha) - 2f_2(\alpha)\,.\,\cos\theta_r\}$
$H_r' = \frac{1}{0\cdot2832n}\{+f_1(\alpha) + 2f_2(\alpha)\,.\,\cos\theta_r\}$</td><td>$\delta_{\text{vertical}} = \frac{R^3}{2EI}\cdot\frac{f_1\alpha}{n}$
$\underset{\text{(in the plane 1-O-1')}}{\delta_{\text{horizontal}}} = \frac{R^3}{EI}\cdot\frac{f_2(\alpha)}{n}$</td></tr>
<tr><td>II</td><td>
</td><td>Vertical reactions
$V_1 = -\frac{1}{2} - \frac{\cos\alpha}{2} + \frac{1}{0\cdot2832n}\{(n-1)f_3(\alpha) - (n-2)f_4(\alpha)\}$
$V_1' = -\frac{1}{2} + \frac{\cos\alpha}{2} + \frac{1}{0\cdot2832n}\{(n-1)f_3(\alpha) + (n-2)f_4(\alpha)\} = -H_1'$
Horizontal reactions
$H_1 = +\frac{1}{2} - \frac{\cos\alpha}{2} + \frac{1}{0\cdot2832n}\{-(n-1)f_3(\alpha) + (n-2)f_4(\alpha)\}$
$H_1' = +\frac{1}{2} - \frac{\cos\alpha}{2} + \frac{1}{0\cdot2832n}\{-(n-1)f_3(\alpha) - (n-2)f_4(\alpha)\}$</td><td>Vertical reactions
$V_r = \frac{1}{0\cdot2832n}\{-f_3(\alpha) + 2f_4(\alpha)\,.\,\cos\theta_r\} = -H_r$
$V_r' = \frac{1}{0\cdot2832n}\{-f_3(\alpha) - 2f_4(\alpha)\,.\,\cos\theta_r\} = -H_r'$
Horizontal reactions
$H_r = \frac{1}{0\cdot2832n}\{+f_3(\alpha) - 2f_4(\alpha)\,.\,\cos\theta_r\}$
$H_r' = \frac{1}{0\cdot2832n}\{+f_3(\alpha) + 2f_4(\alpha)\,.\,\cos\theta_r\}$</td><td>$\delta_{\text{vertical}} = \frac{R^3}{2EI}\cdot\frac{f_3(\alpha)}{n}$
$\underset{\text{(in the plane 1-O-1')}}{\delta_{\text{horizontal}}} = \frac{R^3}{EI}\cdot\frac{f_4(\alpha)}{n}$</td></tr>
</table>

FIG. 10

eaction components at the support 2.

(a) Vertical: $V_2 = \frac{1}{0{\cdot}2832n}\{-f_1(\alpha) + 2f_2(\alpha)\cos\theta_2\}$

$$= \frac{1}{0{\cdot}2832n}\{0{\cdot}0498 + 2 \times 0{\cdot}1498 \times 0{\cdot}7071\} = +0{\cdot}2310 \text{ ton}$$

(b) Horizontal: $H_2 = \frac{1}{0{\cdot}2832n}\{f_1(\alpha) - 2f_2(\alpha)\cos\theta_2\}$

$$= \frac{1}{0{\cdot}2832 \times 4}\{-0{\cdot}0498 - 2 \times 0{\cdot}1498 \times 0{\cdot}7071\}$$

$$= -0{\cdot}2310 \text{ ton}$$

SUMMARY

Vertical reaction components: ton	Horizontal reaction components: ton
$V_1 = -0{\cdot}0428$	$H_1 = -0{\cdot}2500$
$V_2 = 0{\cdot}2310$	$H_2 = -0{\cdot}2310$
$V_3 = 0{\cdot}0440$	$H_3 = -0{\cdot}0440$
$V_4 = -0{\cdot}1431$	$H_4 = 0{\cdot}1431$
$V_1' = -0{\cdot}2209$	$H_1' = 0{\cdot}2209$
$V_2' = -0{\cdot}1431$	$H_2' = 0{\cdot}1431$
$V_3' = 0{\cdot}0440$	$H_3' = -0{\cdot}0440$
$V_4' = 0{\cdot}2310$	$H_4' = -0{\cdot}2310$

a check:

$$\Sigma V = -0{\cdot}5499 + 0{\cdot}5500 \simeq 0$$

e deflexion components of the apex of the dome under this loading, according to e formulae given in Fig. 10, are as follows:

$$\delta_v{}^0 = \frac{R^3}{2EI} \cdot \frac{f_1(\alpha)}{n} = -\frac{R^3}{EI}(0{\cdot}0062) \text{ (upwards)}$$

$$\delta_h{}^0 = \frac{R^3}{EI} \cdot \frac{f_2(\alpha)}{n} = +\frac{R^3}{EI}(0{\cdot}0375) \text{ (towards 1')}$$

e diagrams of the bending moments and shearing and axial forces in all ribs of the me, caused by the considered loading, are given in Fig. 11.

Fig. 12 shows similar diagrams for the same dome under a unit vertical force se II).

EXPERIMENTAL ANALYSIS OF A RIBBED DOME

To verify the developed general formulae, experiments on a model of a simple bed dome were carried out. The model consisted of eight ribs made of 20-gauge ss tube. The span of the model was 3 ft 0 in. All ribs were fitted at their ends h specially prepared hard steel balls resting in bearings. These latter allowed ooth rotation of the balls, but prevented any movement of the supports. All the ht ribs were connected at the apex to a common ball by means of threaded thin s, all ribs intersecting at the same point. The moment of inertia of the thin pins very small compared with that of the tube; the pins were thus very flexible, sting only a very small amount of moment. This connexion acted almost as a erical hinge.

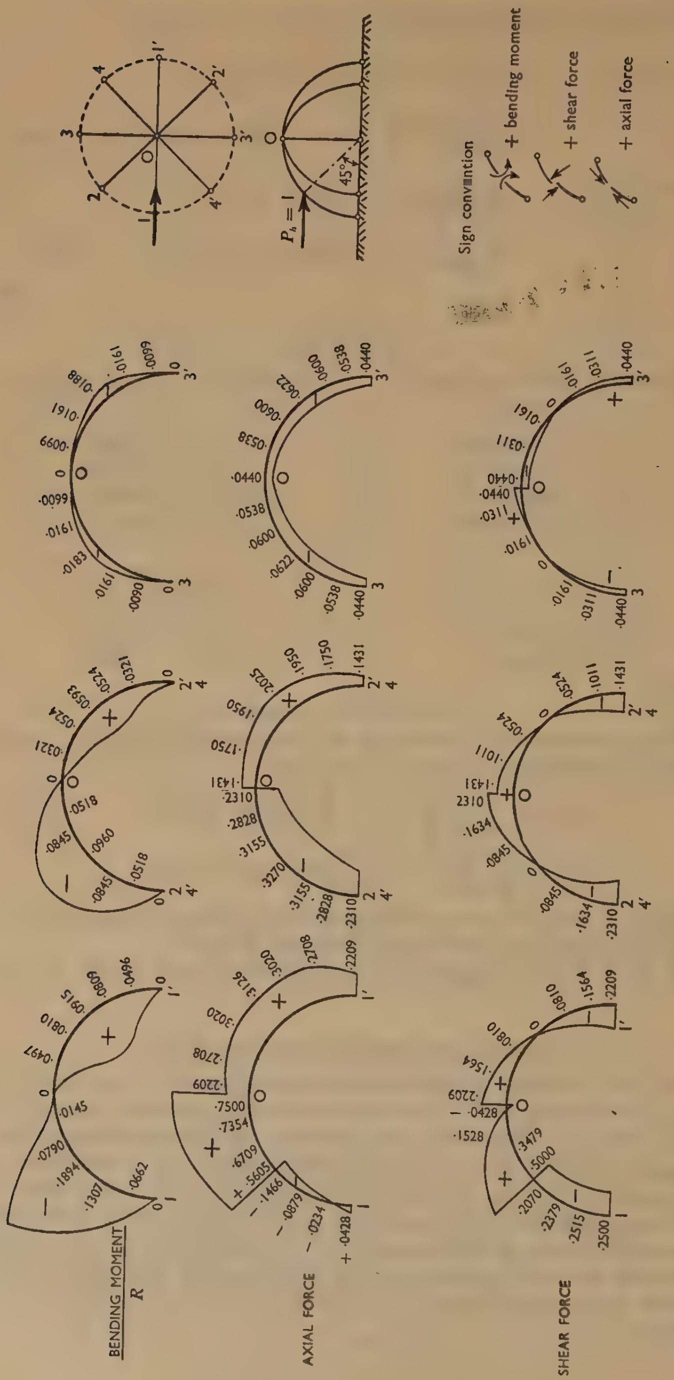

$P_h = 1$
45°
Sign convention
+ bending moment
+ shear force
+ axial force
BENDING MOMENT / R
AXIAL FORCE
SHEAR FORCE

FIG. 11

Fig. 12

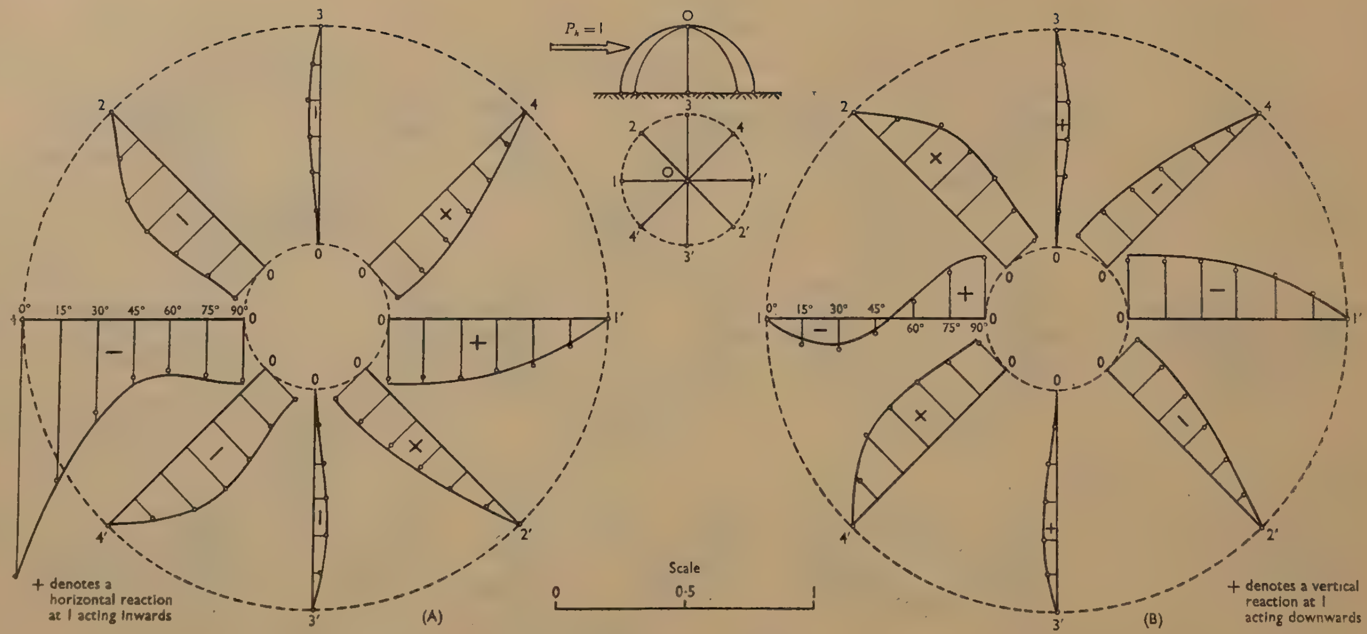


FIG. 13.—INFLUENCE LINES FOR (A) HORIZONTAL AND (B) VERTICAL REACTIONS AT SUPPORT 1 FOR A UNIT HORIZONTAL FORCE ACTING AT ANY ANGLE α ON ANY RIB

(Points are plotted from experimental values; curves from calculated values)

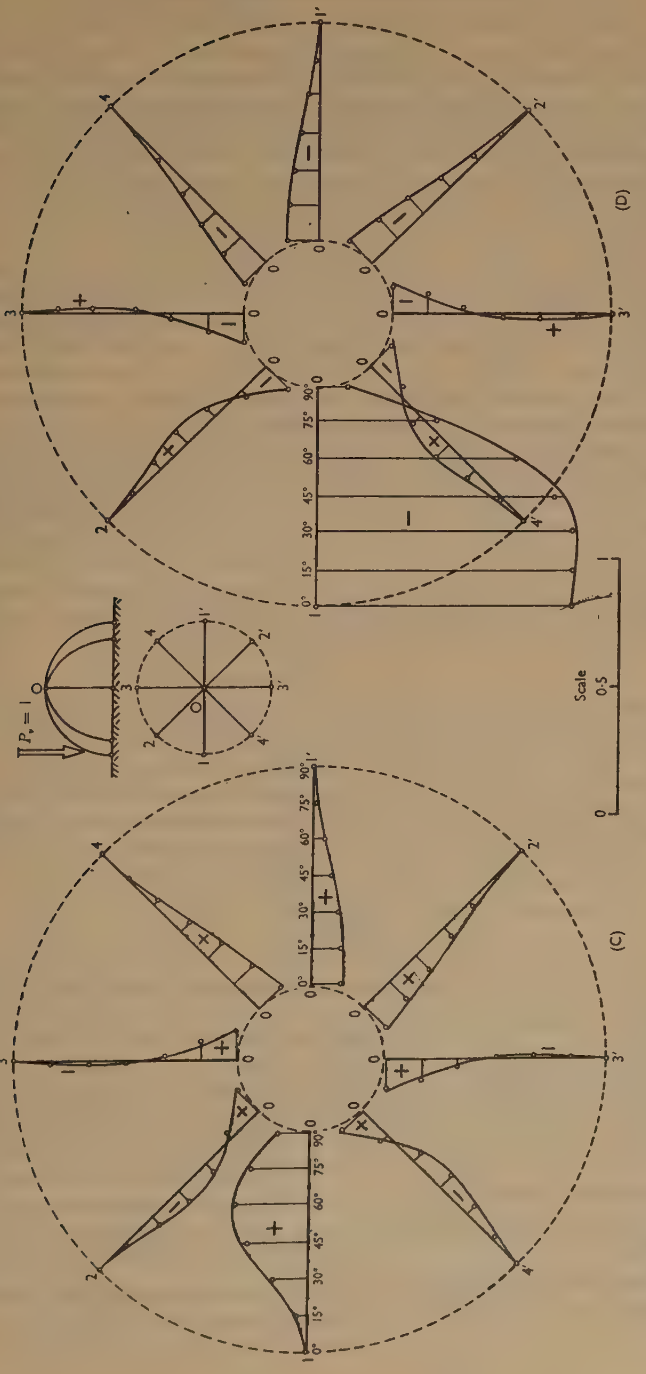


FIG. 14.—INFLUENCE LINES FOR (C) HORIZONTAL AND (D) VERTICAL REACTIONS AT SUPPORT 1 FOR A UNIT VERTICAL FORCE ACTING AT ANY ANGLE α ON ANY RIB

The direct method of loading was used first. Horizontal and vertical loads were applied at different points and the resulting displacements of the apex of the dome measured by cathetometers. The experimental displacements compared favourably with the analytical ones. The indirect method, based on Clerk-Maxwell's reciprocal theorem, was then applied. It is known that if the support at any point r is released and a displacement Δ is imposed in, say, a vertical direction so that it has no component displacements in any other directions, then, if any point M of the dome is thereby displaced by δ in, say, a horizontal direction, by Clerk-Maxwell's theorem it can be stated that a horizontal load P, acting at M will produce a vertical reaction at r equal to $-P\delta/\Delta$. Because of the cyclic symmetry of the dome it is necessary to move only one support in order to obtain the influence lines for reaction components for all supports. The horizontal and vertical displacements were introduced at such a support by a special space deformeter, on which the ball joint of the considered rib had been mounted. The indirect method and the space deformeter are fully described in a previous Paper[10] on the experimental analysis of space structures.

The indirect method proved to be quick and very efficient in the determination of the influence lines of the reactions. Figs 13 and 14 show the influence lines of reaction components for the support 1 for horizontal and vertical loads acting on the dome. The agreement between the experimental and analytical values obtained by the model and by the developed theory respectively is generally satisfactory, verifying the validity of the theory. In several cases the experimental values are slightly smaller than the calculated ones. This discrepancy is probably caused by a small amount of fixity existing in the model at the apex connexion and at the supports, which theoretically should be frictionless spherical hinges.

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The Paper, which was received on 8 June, 1956, is accompanied by one photograph and six sheets of diagrams, from some of which the Figures in the text have been prepared.

Correspondence on this Paper should be forwarded to reach the Institution before 15 March, 1957. Contributions should not exceed 1,200 words.—Sec.

Paper No. 6153

HE SOLUTION OF REDUNDANT FRAMEWORKS BY SUCCESSIVE REDUCTION OF THE GAPS IN CUT MEMBERS

by

* William Wright, B.Sc., Ph.D., A.M.I.C.E.

(*Ordered by the Council to be published with written discussion*)

Introduction

IE method outlined is applicable to pin-jointed redundant frameworks, and though the technique used is relaxational the physical concept on which it rests—gap reduction"—is quite different from that used in the orthodox solution by laxation methods [1]—the liquidation of unbalanced forces.

Nomenclature

forces	The forces arising in the members of a structure, due to the application of an external load W.
$= \dfrac{L}{AE}$	The change in length of a member due to the application of unit load to the member.
G_2, G_3, etc.	The sizes of the gaps in cut members, 1, 2, 3, etc. respectively. A negative sign denotes separation of the cut ends, a positive sign overlapping of the cut ends. See Fig. 1.
, U_2, U_3, etc.	The forces arising in members due to the application of unit tensile load across gaps 1, 2, 3, etc. respectively. See Fig. 1.
T_2, T_3, etc.	The forces necessary to close gaps 1, 2, 3, etc. respectively, i.e., the final forces in the cut members.

Method

Consider the structure shown in Fig. 1. Six redundant members must be imagined be cut to give a statically determinate structure; the selected members are those nbered 1 to 6. When W is applied the resulting gaps in the cut members G_1, G_2, etc. will be respectively $\Sigma U_1 We$, $\Sigma U_2 We$, $\Sigma U_3 We$, etc. by the well-known formula. hould be noted that these summations do not include the cut members.

The problem now reduces to that of finding the necessary forces T_1, T_2, T_3, etc. to applied to the cut members so that, in general, all gaps will just close. This essitates calculating the amount by which each gap in turn is changed by the lication of unit load across the gap and also the effect of this operation on adjacent s. This effect can be expressed concisely by the following formula: change in

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* The Author is Head of the Civil Engineering Department, The University, Southton.

[1] R. V. Southwell, "Relaxation Methods in Engineering Science". Oxf. Univ. Press,).

TABLE 1

Member	(1) $e \times 10^6$	(2) W forces	(3) U_1	(4) U_2	(5) U_3	(6) U_4	(7) U_5	(8) U_6	(9) U_1We	(10) U_2We	(11) U_3We	(12) U_4We	(13) U_5We	(14) U_6We	(15) Final forces
1	2	0	+ 1												− 56·9
2	2	0		+ 1											− 35·4
3	2	0			+ 1										+ 33·8
4	2	0				+ 1									− 25·3
5	2	0					+ 1								− 22·8
6	2	0						+ 1							− 27·6
7	2	+ 94·3	+ 1						+ 188·6						+ 37·4
8	2	+ 94·3		+ 1						+ 188·6					+ 58·9
9	2	− 47·1			+ 1						− 94·3				− 13·5
10	2	+ 47·1				+ 1						+ 94·3			+ 21·8
11	2	+ 47·1					+ 1						+ 94·3		+ 24·3
12	2	+ 47·1						+ 1						+ 94·3	+ 19·5
13	2·5	− 66·7	− 0·707						+ 117·8						− 26·7
14	2·5	− 66·7	− 0·707	− 0·707					+ 117·8	+ 117·8					− 1·7
15	2·5	+ 33·3		− 0·707	− 0·707					− 58·9	− 58·9				+ 34·5
16	2·5	0			− 0·707	− 0·707					0	0			− 5·9
17	2·5	− 33·3				− 0·707	− 0·707					+ 58·9	+ 58·9		+ 0·7

TABLE 1—*continued*

Member	(1) $e \times 10^6$	(2) W forces	(3) U_1	(4) U_2	(5) U_3	(6) U_4	(7) U_5	(8) U_6	(9) U_1We	(10) U_2We	(11) U_3We	(12) U_4We	(13) U_5We	(14) U_6We	(15) Final forces
18	2·5	− 33·3					− 0·707	− 0·707					+ 58·9	+ 58·9	+ 2·3
19	2·5	− 33·3						− 0·707						+ 58·9	− 13·8
20	1	− 66·7	− 0·707						+ 47·1						− 26·7
21	1	− 133·3		− 0·707						+ 94·3					− 108·3
22	1	− 100			− 0·707						+ 70·7				− 123·8
23	1	− 100				− 0·707						+ 70·7			− 82·1
24	1	− 66·7					− 0·707						+ 47·1		− 50·6
25	1	− 33·3						− 0·707						+ 23·5	− 13·8
26	1	0	− 0·707						0						+ 40
27	1	+ 66·7		− 0·707						− 47·1					+ 91·7
28	1	+ 133·3			− 0·707						− 94·3				+ 109·5
29	1	+ 66·7				− 0·707						− 47·1			+ 84·6
30	1	+ 33·3					− 0·707						− 23·6		+ 49·4
31	1	0						− 0·707						0	+ 19·5
								Σ	+ 471·3	+ 294·7	− 176·8	+ 176·8	+ 235·6	+ 235·6	Initial Gaps

TABLE 2

Operation	Effect on G_1	Effect on G_2	Effect on G_3	Effect on G_4	Effect on G_5	Effect on G_6
(1) $T_1 = +1$	$\Sigma U_1^2 e = 7{\cdot}5$	$\Sigma U_1 U_2 e = 1{\cdot}25$	$\Sigma U_1 U_3 e = 0$	$\Sigma U_1 U_4 e = 0$	$\Sigma U_1 U_5 e = 0$	$\Sigma U_1 U_6 e = 0$
(2) $T_2 = +1$	$\Sigma U_2 U_1 e = 1{\cdot}25$	$\Sigma U_2^2 e = 7{\cdot}5$	$\Sigma U_2 U_3 e = 1{\cdot}25$	$\Sigma U_2 U_4 e = 0$	$\Sigma U_2 U_5 e = 0$	$\Sigma U_2 U_6 e = 0$
(3) $T_3 = +1$	$\Sigma U_3 U_1 e = 0$	$\Sigma U_3 U_2 e = 1{\cdot}25$	$\Sigma U_3^2 e = 7{\cdot}5$	$\Sigma U_3 U_4 e = 1{\cdot}25$	$\Sigma U_3 U_5 e = 0$	$\Sigma U_3 U_6 e = 0$
(4) $T_4 = +1$	$\Sigma U_4 U_1 e = 0$	$\Sigma U_4 U_2 e = 0$	$\Sigma U_4 U_3 e = 1{\cdot}25$	$\Sigma U_4^2 e = 7{\cdot}5$	$\Sigma U_4 U_5 e = 1{\cdot}25$	$\Sigma U_4 U_6 e = 0$
(5) $T_5 = +1$	$\Sigma U_5 U_1 e = 0$	$\Sigma U_5 U_2 e = 0$	$\Sigma U_5 U_3 e = 0$	$\Sigma U_5 U_4 e = 1{\cdot}25$	$\Sigma U_5^2 e = 7{\cdot}5$	$\Sigma U_5 U_6 e = 1{\cdot}25$
(6) $T_6 = +1$	$\Sigma U_6 U_1 e = 0$	$\Sigma U_6 U_2 e = 0$	$\Sigma U_6 U_3 e = 0$	$\Sigma U_6 U_4 e = 0$	$\Sigma U_6 U_5 e = 1{\cdot}25$	$\Sigma U_6^2 e = 7{\cdot}5$
(7) $T_1 = T_2 = T_3 = T_4 = T_5 = T_6 = 1$	8·75	10	10	10	10	8·75
(8) $T_1 = 24;\ T_2 = -4$	175	0	− 5	0	0	0
(9) $T_1 = T_3 = -4;\ T_2 = 24$	0	170	0	− 5	0	0
(10) $T_2 = T_4 = -4;\ T_3 = 24$	− 5	0	170	0	− 5	0
(11) $T_3 = T_5 = -4;\ T_4 = 24$	0	− 5	0	170	0	− 5
(12) $T_4 = T_6 = -4;\ T_5 = 24$	0	0	− 5	0	170	0
(13) $T_5 = -4;\ T_6 = 24$	0	0	0	− 5	0	175

gap r due to application of unit load across gap $n = \Sigma U_n U_r e$. Thus the effect of unit load applied across gap 1 ($n = 1$) on gap 1 ($r = 1$) is $\Sigma U_1^2 e$ and on gap 2 ($r = 2$) is $\Sigma U_1 U_2 e$ and so on. It should be noted that the summation includes the cut redund-ant members and that, by Maxwell's Reciprocal Theorem, $\Sigma U_n U_r e = \Sigma U_r U_n e$.

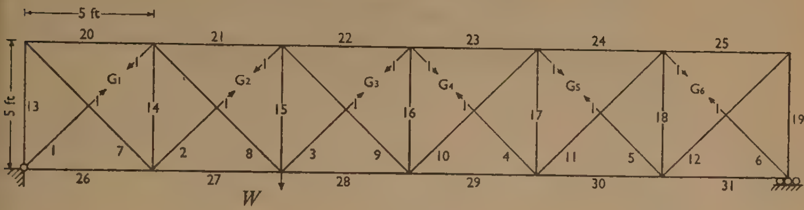

FIG. 1

The procedure is best illustrated in the following example:

Find the forces in the members of the frame in Fig. 1 with $W = 100$ lb.; area of chords $= 2$ sq. in.; $E = 30 \times 10^6$ lb/sq. in.; area of verticals $= 0{\cdot}8$ sq. in.; area of diagonals $= \sqrt{2}$ sq. in.

TABULATION

Column 1 Gives the extensions of the members under unit load. The actual extensions have been multiplied by 10^6 for convenience in the arithmetical work.

Column 2 Gives the forces in the statically determinate structure due to W.

Column 3 Gives the forces due to the application of unit load across gap 1. It should be noted that only 6 members are affected.

Columns 4–8 Show the effect of unit load across the remaining gaps in turn.

Column 9 Gives the U_1We terms, the sum of which gives G_1 the gap in member 1 due to W. Strictly speaking there is an overlap and not a gap since the sum of the terms is positive.

Columns 10–14 Show the computations for gaps 2–6.

Table 2 shows the extent to which the gaps are reduced by the application of unit load across each gap in turn.

The first six lines of Table 2 are computed from the data in Table 1 and show the effect of applying unit load in turn across each gap. For example, in line 1 the effect of unit load across gap 1 is to decrease gap 1 by $\Sigma U_1^2 e = 7{\cdot}5$ units and gap 2 by $\Sigma U_1 U_2 e = 1{\cdot}25$ units; the other gaps are unaffected. Line 7 shows the effect of applying unit loads across all six gaps at the same time. The effect of the group operators shown in lines 8–13 is, to all intents and purposes, confined to one gap at a time and their use greatly speeds up convergence; they are easily calculated from the basic unit operators in lines 1–6.

Table 3—the gap reduction Table—records the operations required to bring the gaps to zero or negligible values.

The first line of the Table gives the initial values of the gaps due to W only, *i.e.*, the values with $T_1 = T_2 = T_3 = T_4 = T_5 = T_6 = 0$.

The operation in the second line which is operator No. 7 in Table 2 multiplied by 25 is designed to make the sum of the six gaps very approximately zero.

TABLE 3

Operation	G_1	G_2	G_3	G_4	G_5	G_6
$T_1 = T_2 = T_3 = T_4 = T_5 = T_6 = 0$	+ 471	+ 295	− 177	+ 177	+ 236	+ 236
$T_1 = T_2 = T_3 = T_4 = T_5 = T_6 = -25$	+ 252	+ 45	− 427	− 73	− 14	+ 17
$T_2 = T_4 = -10$; $T_3 = 60$	+ 240	+ 45	− 2	− 73	− 26	+ 17
$T_1 = -32{\cdot}88$; $T_2 = 5{\cdot}48$	0	+ 45	+ 5	− 73	− 26	+ 17
$T_3 = T_5 = -1{\cdot}72$; $T_4 = 10{\cdot}32$	0	+ 43	+ 5	0	− 26	+ 15
$T_1 = T_3 = 1$; $T_2 = -6$	0	0	+ 5	+ 1	− 26	+ 15
$T_4 = T_6 = -0{\cdot}6$; $T_5 = 3{\cdot}6$	0	0	+ 4	+ 1	0	+ 15
$T_5 = 0{\cdot}36$; $T_6 = -2{\cdot}16$	0	0	+ 4	+ 1	0	− 1
$T_3 = -0{\cdot}5$	0	− 1	0	0	0	− 1
$T_2 = T_6 = 0{\cdot}13$	0	0	0	0	0	0
ΣT: $T_1 = -56{\cdot}9$; $T_2 = -35{\cdot}4$; $T_3 = +33{\cdot}8$; $T_4 = -25{\cdot}3$; $T_5 = -22{\cdot}8$; $T_6 = -27{\cdot}6$						

The next operation, which is operation No. 10 in Table 2, multiplied by 2·5 is designed to reduce gap 3 approximately to zero.

The process is continued until all the gaps are reduced to zero or as near to zero as desired. It should be noted that attention is concentrated all the time on the current gap sizes and it is those which are recorded in the Table (correct to the nearest whole number). The individual operators are now summed to give the final values of T_1 to T_6. These are the forces which must be applied to the cut members to reduce all gaps to zero, i.e., they are the required forces in the redundant members. Once they are known the forces in the other members can be calculated and inserted in column 15 of Table 1.

CONCLUSIONS

The method gives the actual forces in the members of a structure directly and with relatively little labour. It is particularly useful in solving problems involving "lack of fit" and could easily be applied to space frames.

The Paper, which was received on 30 May, 1956, is accompanied by one diagram, from which the Figure in the text has been prepared.

CORRESPONDENCE should be forwarded to reach the Institution before 15 March, 1957. Contributions should be limited to about 1,200 words.—SEC.

CORRESPONDENCE

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Paper No. 6055

A general method for the analysis of grid frameworks †

by

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and

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Ir S. Kendrick (Senior Scientific Officer, Naval Construction Research Establish-nt, Dunfermline) observed that the subject of grillage analysis had an extensive rature but torsional effects were usually neglected or treated extremely approximately, for example, in the orthotropic plate approach by which torsional rigidity was easily wed for. The Paper attempted a more complete analysis and appeared to be very cessful in obtaining theoretical results which agreed with experiment. That agree-nt seemed, however, to be more coincidental than a reflexion of the intrinsic accuracy the theory. The main justification for that statement came from exact analyses sfying the relaxation equations which Mr Kendrick had carried out for the grillage wn in Fig. 3a using the following three different physical assumptions:

(*a*) zero torsional stiffness of both longitudinal and transverse beams (the most usual physical assumption);

(*b*) zero torsional stiffness of the transverse beams and finite torsional stiffness of the longitudinal beams;

(*c*) zero torsional stiffness of the transverse beams with no rotation in the longitudinal beams.

he following structural parameters, sufficient to define the problem, were deduced the numerical example in the Paper:

$I_2/I_1 = 2{\cdot}9$, 11·5/3·97 from p. 953; $I_1/I_T = 2{\cdot}06$ from $\alpha = 11{\cdot}5$, p. 953; CJ_1/EI_T $= 2{\cdot}80$, from $\beta' = 1{\cdot}15$, p. 952; $CJ_2/EI_T = 8{\cdot}27$ from $\beta = 3{\cdot}40$, p. 952; W/EI_1 $= 0{\cdot}000374$ in^{-2} from $WL^3/9EI_1 = 1{\cdot}94$, p. 954.

ie values for intersection deflexions obtained by those analyses were given in Table 2, ; the notation that the point *rs* was the intersection of the *r*th longitudinal with the ransverse beam (measured from the left).

† Proc. Instn Civ. Engrs, Pt III, vol. 4, p. 939 (Dec. 1955).

TABLE 2.—INTERSECTION DISPLACEMENTS

Point	Assumption (a): in.	Assumption (b): in.	Assumption (c): in.	Hendry and Jaeger Theory (full): in.	Hendry and Jaeger Experiment: in.
11	0·118	0·0978	0·0416	0·076	0·080
12	0·206	0·161	0·0640	0·131	0·127
13	0·0887	0·0730	0·0223	0·055	0·050
21	0·0303	0·0277	0·0230	0·027	0·024
22	0·0588	0·0538	0·0426	0·050	0·049
23	0·0285	0·0251	0·0196	0·023	0·024
31	− 0·00575	0·000388	0·0163	0·006	0·007
32	− 0·0105	0·00269	0·0320	0·012	0·012
33	− 0·00476	− 0·000653	0·0157	0·006	0·005
41	− 0·0181	− 0·00849	0·0149	− 0·002	− 0·001
42	− 0·0365	− 0·0150	0·0295	− 0·003	− 0·003
43	− 0·0184	− 0·00841	0·0146	− 0·002	− 0·002

Also included in Table 2 were the theoretical and experimental deflexions given in t[he]
Paper.

The physical assumptions used by the Authors in the full theory were identical wi[th]
those of assumption (b). Also, the values labelled assumption (b) in Table 2 were exa[ct]
to three figures (calculated by simultaneous-equation solution using six significant figure[s].)
Thus the differences between the Authors' theoretical values and those labelled assum[p]tion (b) represented the errors in the former values and were approximately 25% for t[he]
larger deflexions. The agreement between the Authors' theoretical and experiment[al]
values (3% for the largest deflexion) was thus seen to be a remarkable coincidence a[nd]
not due to their theory correctly accounting mathematically for correct physical assum[p]tions.

Mr Kendrick believed that the errors in the Authors' theoretical values probably aro[se]
from the incomplete and incorrect treatment of the longitudinal rotations. That ha[d]
the effect of exaggerating the torsional stiffness and thereby reducing the deflexio[ns.]
The incorrectness of the treatment of θ lay in the use of the integral conditions:

$$\int_{-L/2}^{L/2}(M_{10} + M_{12})dx = 0, \text{ etc.}$$

$$CJ_1[\theta_1]_0^{L/2} = \int_0^{L/2}\int_0^x (M_{10} + M_{12})dx \, . \, dx, \text{ etc.}$$

instead of a differential equation condition such as:

$$CJ_1 \frac{d^2\theta_1}{dx^2} = M_{10} + M_{12}$$

Those integral conditions led to equations in which all harmonics a, c, and λ appropria[te]
to a particular longitudinal occurred simultaneously. The neglect in those equations [of]
harmonics of order higher than the first was unexplained and in fact unjustifiable. A[lso]
unexplained was the method by which higher harmonic terms in λ could be evaluat[ed]
since no additional equations would be forthcoming. In short, the most vital part of t[he]
Authors' analysis was erroneous.

The lack of agreement between the correct values based on assumption (b) and t[he]
experimental values could arise from several causes. First, the beam stiffnesses used [in]
the calculations might be in error and large inconsistencies between the stated beam si[...]

l the flexural stiffnesses used in the calculation made that possible. For example, ing the applied load as 1 lb., as stated in Fig. 3a, the following values were obtained flexural rigidity (lb./in^2).

	EI_1	EI_2	EI_T
Based on rectangles stated in Fig. 3a	4,890	16,500	2,450
Calculated from numerical example constants .	2,670	7,750	1,290

n view of those very large inconsistencies it would be of considerable interest to know measured values of EI mentioned in Appendix III and the explanation of their parent lack of agreement with the calculated EI values.

Another possible explanation of the lack of agreement between the experimental and rect theoretical values was that the assumed conditions of simple support were not ieved with sufficient accuracy experimentally. The use of rollers in combination with ghts might possibly have introduced end constraint.

A curious feature of the Paper was that there was no discussion of the errors introduced assuming zero torsional rigidity, a very common assumption, although a discussion of errors resulting from the assumption of infinite torsional rigidity, a most uncommon umption, was included. The assumption of zero torsional rigidity led to negligible ors in most practical grillages and greatly simplified the analysis, since the twist θ was n expressible in terms of y. That simplification appeared to have been overlooked the Authors since on p. 956 they inferred that a Fourier expansion for θ was still essary when the torsional resistance of the longitudinals was zero. For the grillage of . 3a torsional rigidity was significant owing to the use of rectangular sections, but ole 2 showed that, even for that grillage, the neglect of torsional rigidity led to no greater or than did the Authors' theory. It was interesting that the "assumption (c)" results Table 2 agreed exactly with those of the Authors, for the case $\lambda = \gamma = c = 0$.

t was difficult to follow the reasoning on p. 947 concerning bending-moment evaluation e the method suggested led to exactly the same series expansion for bending moments vere obtained by differentiating twice the deflexion series. Experimental values for ding stresses, if measured, would be of considerable interest since the bending-nent values of Fig. 3b differed appreciably from the values obtained in the exact tion referred to above.

he amount of experimental work carried out was impressive and a fuller description hat aspect would be valuable.

Ir P. K. Chaudhuri (Research Student, Department of Civil Engineering, Imperial ege of Science and Technology) said the Authors' solution of the interconnected ge-girder problem was, apart from the classical, relaxation, and moment-distribution hods, the only complete method yet developed for plane grid frames which could account of all actual conditions of flexural and torsional restraints on the structure. Authors had provided a valuable yet fairly simple solution of a difficult problem ch, until recently, had probably attracted more attention on the Continent than in ain.

he Authors did not mention how their method compared, in respect of the degree of racy and of the time consumed, with other existing practical methods (which, inci-ally, were of more limited application no doubt). A comparison of that sort might f interest to engineers who went in for the design of grids. The tabulated results of analysis of a particular grid by several practical methods, including the Authors', nt therefore be of value. The idealized structure (described below) chosen for the parison was only theoretical and not designed for any particular loading. Attention been limited to the problem of the grid supported on two sides and only deflexions lated to provide a basis for the comparison of the different methods.

n interconnected reinforced concrete bridge-girder system consisting of four main ers and nine intermediate cross-girders with two end cross-girders (see Fig. 11 for ensions) was analysed under the action of a concentrated load of 10 tons applied at

a quarter-point on an outer girder. The deflexions tabulated later did not include th effect of the dead-weight.

Outer main girders AA, A_1A_1: 2nd moment of area 120,000 in^4; Torsion constan 98,500 in^4

Inner main girders BB, B_1B_1: 2nd moment of area 70,000 in^4; Torsion constant 48,000 in

Cross-girders: 2nd moment of area 10,000 in^4; Torsion constant 4,000 in^4

Assumed modulus of elasticity $E = 3 \times 10^6$ lb/sq. in.

Modulus of rigidity of concrete $G = \dfrac{E}{2(1+\mu)} = \dfrac{E}{2(1+0{\cdot}1)} = 0{\cdot}455E$

The longitudinal main girders were simply supported at their ends.

The structure was analysed by the Authors' method for four cases:—

Case 1. A full analysis taking all the above properties into account.

Case 2. An analysis from which the effect of the torsion of the transverse mediu was omitted (i.e., with $\gamma = 0$).

Case 3. An analysis "omitting all torsional and rotational effects" i.e., wit $\lambda = c = \gamma = 0$; in other words, assuming that the girders remained upright and di not twist.

Case 4. An analysis for a pin-connected structure with the longitudinals possessin zero torsional resistance.

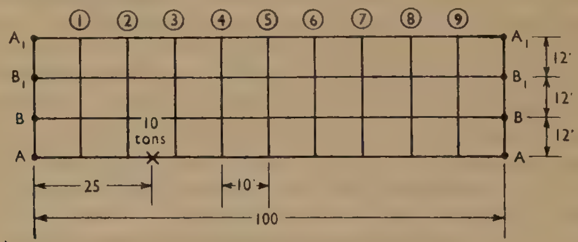

FIG. 11

Deflexions were then obtained for the node points by several other methods. It wou be noticed that Professor Pippard's method assumed that the longitudinals remain upright (cf. Case 3) and Leonhardt's method [9] for pin-connected structures of negligil torsional rigidity corresponded to Case 4 above. A direct comparison with Guyon's and Massonnet's [3] methods was not possible since those methods could not take accou of the extra stiffness of the edge beams. For a rough comparison, however, a simi structure with four identical main girders, each having second moment of area 95,000 and torsion constant 71,000 in^4, i.e., more or less a mean of the two sizes in the structu under consideration, was analysed by Guyon's "no-torsion" method (cf. Case 4) a Massonnet's method taking the torsional resistance of the girders into considerat (cf. Case 1). The results were given in Tables 3–10.

A perusal of Tables 3–10 would indicate clearly that within the limitations of th assumptions the different methods tallied quite well. That was especially true of maximum values, as would be noticed if Table 3 was compared with Table 10, Tabl with Table 7, and Table 6 with Tables 8 and 9. Since grids were nearly always desig as symmetrical structures with girders of uniform section the maximum values were most importance in design. Hence the discrepancy in some values for girder A_1 Tables 6, 8, and 9 probably need not be viewed with particular concern. The values Tables 3 and 4 showed clearly that the torsion of the transverse medium might safely omitted from the analysis—a conclusion already reached by the Authors. Furtherm

[9] References 9–18 are on p. 864.

comparison between Tables 3 and 6 would lead to the interesting observation that even ı the present case, where the reinforced concrete girders possessed high torsional rigidities, ıe analysis of the structure as a pin-connected one composed of torsionally weak members id not give results very different from the actual. In an interconnected steel bridge irder, therefore, the much simpler pin-connected analysis was quite adequate.

On the question of time and labour consumed in the different analyses, Mr Chaudhuri's xperience was that Leonhardt's method was much simpler and quicker in application ıan the Authors' and hence, wherever admissible, i.e., in steel bridge-girder cases as xplained above, should be preferred to the latter for design-office use. The same also eld for Guyon's "no-torsion" and Massonnet's "torsion" analyses provided, of course, ıe main girders in a bridge system were of identical cross-section. If, in addition, the beam and load positions" happened to coincide with those for which the Guyon and ſassonnet charts were available then those two were undoubtedly the quickest methods ɔr the solution of interconnected bridge-girder systems under the respective assumptions. onsidering, however, bridge-beam systems with longitudinal girders of varying sizes and ossessing a high torsional rigidity the value of the Authors' method at once became ıanifest. Apart from the classical, relaxation, and moment-distribution methods, all of hich were too lengthy to be applied to such structures, the Authors' method was the nly one available for use in an average design office for the solution of such structures. 'o that must be added the flexibility of the method which made its extension possible to rids supported on more than two sides, as the Authors had shown.

Tables 3–10 also endorsed the Authors' remark that the assumption that the longiudinals did not rotate (i.e., the Pippard assumption) was quite inadequate. Besides, the ery long and cumbersome calculations involved in the Pippard solution left it out of the verage designer's reckoning.

In that connexion the expression "analysis omitting all torsional and rotational effects", hich the Authors used (e.g., in the example of Fig. 3) to denote the analysis wherein $= c = \gamma = 0$ did not seem to be very felicitous. The expression had a chance of being ıisinterpreted as an analysis in which the torsional and rotational restraints of the

TABLES OF DEFLEXIONS (EVALUATED AT NODES ONLY)
(All values in inches)

TABLE 3.—THE AUTHORS' METHOD—CASE 1 (FULL ANALYSIS)
(Compare with Table 10)

ɟirder	L.H.E.	1	2	3	4	5	6	7	8	9	R.H.E.
A	0	0·3320	0·6210	0·8360	0·9550	0·9750	0·8940	0·7400	0·5250	0·2720	0
B	0	0·2330	0·4370	0·5800	0·6560	0·6630	0·6025	0·5175	0·3460	0·1790	0
B_1	0	0·1384	0·2575	0·3420	0·3840	0·3860	0·3460	0·2810	0·1959	0·1002	0
A_1	0	0·0631	0·1161	0·1510	0·1650	0·1590	0·1370	0·1061	0·0717	0·0354	0

TABLE 4.—THE AUTHORS' METHOD—CASE 2 ($\gamma = 0$)
(Compare with Table 3)

irder	L.H.E.	1	2	3	4	5	6	7	8	9	R.H.E.
A	0	0·3260	0·6125	0·8240	0·9375	0·9560	0·8760	0·7275	0·5155	0·2650	0
B	0	0·2270	0·4180	0·5675	0·6405	0·6460	0·5870	0·4805	0·3319	0·1739	0
B_1	0	0·1318	0·2450	0·3240	0·3640	0·3630	0·3260	0·2630	0·1830	0·0935	0
A_1	0	0·0551	0·1011	0·1305	0·1530	0·1331	0·1150	0·0859	0·0561	0·0273	0

TABLE 5.—THE AUTHORS' METHOD—CASE 3 ($\lambda = c = \gamma = 0$)
(Compare with Table 7)

Girder	L.H.E.	1	2	3	4	5	6	7	8	9	R.H.E.
A	0	0·1975	0·3640	0·4840	0·5400	0·5375	0·4810	0·3845	0·2895	0·1381	0
B	0	0·1880	0·3480	0·4635	0·5395	0·5200	0·4675	0·3775	0·2610	0·1371	0
B_1	0	0·1760	0·3280	0·4370	0·4980	0·5040	0·4600	0·3765	0·2650	0·1341	0
A_1	0	0·1660	0·3120	0·4210	0·4840	0·4925	0·4550	0·3760	0·2675	0·1360	0

TABLE 6.—THE AUTHORS' METHOD—CASE 4 (PIN-CONNECTED STRUCTURE)
(Compare with Tables 8 and 9)

Girders	L.H.E.	1	2	3	4	5	6	7	8	9	R.H.E.
A	0	0·4200	0·7890	1·065	1·220	1·251	1·160	0·9700	0·6910	0·3570	0
B	0	0·2450	0·4500	0·6150	0·6950	0·7040	0·6400	0·5275	0·3640	0·1915	0
B_1	0	0·0860	0·1585	0·2030	0·2230	0·2150	0·1860	0·1434	0·0960	0·0476	0
A_1	0	− 0·0578	− 0·1140	− 0·1650	− 0·2060	− 0·2310	− 0·2340	− 0·2100	− 0·1590	− 0·0855	0

TABLE 7.—PIPPARD'S METHOD
(Compare with Table 5)

Girder	L.H.E.	1	2	3	4	5	6	7	8	9	R.H.E.
A	0	0·2026	0·3739	0·4799	0·5129	0·4961	0·4398	0·3585	0·2522	0·1301	0
B	0	0·1832	0·3365	0·4457	0·4938	0·4891	0·4408	0·3594	0·2532	0·1301	0
B_1	0	0·1710	0·3200	0·4241	0·4782	0·4811	0·4385	0·3598	0·2543	0·1315	0
A_1	0	0·1633	0·3076	0·4114	0·4686	0·4754	0·4343	0·3598	0·2550	0·1321	0

TABLE 8.—LEONHARDT'S METHOD
(Compare with Tables 6 and 9)

Girder	L.H.E.	1	2	3	4	5	6	7	8	9	R.H.E.
A	0	0·4608	0·9130	1·1052	1·2432	1·2432	1·1568	0·9360	0·4140	0·2064	0
B	0	0·2424	0·4392	0·5868	0·6456	0·6456	0·5868	0·4800	0·3360	0·1680	0
B_1	0	0·0732	0·1308	0·1692	0·1896	0·1848	0·1656	0·1380	0·0960	0·0504	0
A_1	0	− 0·0730	− 0·1380	− 0·2076	− 0·1980	− 0·1944	− 0·1776	− 0·1428	− 0·1032	− 0·0516	0

TABLE 9.—GUYON'S METHOD
(Compare with Tables 6 and 8)

Girder	L.H.E.	1	2	3	4	5	6	7	8	9	R.H.E.
A	0	0·5100	0·9360	1·231	1·315	1·354	1·220	1·0000	0·7100	0·3750	0
B	0	0·2695	0·4946	0·6500	0·6925	0·7145	0·6440	0·5280	0·3740	0·1972	0
B_1	0	0·0661	0·1212	0·1597	0·1705	0·1750	0·1580	0·1296	0·0920	0·0485	0
A_1	0	− 0·1061	− 0·1950	− 0·2570	− 0·2740	− 0·2820	− 0·2540	− 0·2085	− 0·1475	− 0·0780	0

TABLE 10.—MASSONNET'S METHOD
(Compare with Table 3)

Girder	L.H.E.	1	2	3	4	5	6	7	8	9	R.H.E.
A	0	0·3285	0·6040	0·7900	0·8450	0·8740	0·7850	0·6460	0·4560	0·2410	0
B	0	0·2200	0·4050	0·5310	0·5680	0·5850	0·5260	0·4310	0·3060	0·1602	0
B_1	0	0·1250	0·2300	0·3020	0·3220	0·3320	0·2990	0·2450	0·1735	0·0916	0
A_1	0	0·0515	0·0945	0·1240	0·1320	0·1360	0·1230	0·1005	0·0715	0·0377	0

structure were not taken into account, i.e., an analysis of a pin-connected structure composed of girders of negligible torsional stiffness, which was the opposite of what was intended. Mr Chaudhuri suggested the expression "an analysis assuming that no torsional and rotational strains are imposed on the girders" as a substitute.

Mr Chaudhuri pointed out that the effect of the disposition of the transverse girders along the span of the bridge was more important in steel bridges of low torsional rigidity than in concrete ones.

Mr G. Sved (Senior Lecturer in Strength of Materials) and **Mr D. Brooks** (Research Student), of the Department of Civil Engineering, University of Adelaide, observed that the Paper contained a valuable contribution to the solution of an important practical problem. There were, however, some points on which they wished to comment in the light of investigations carried out at the University of Adelaide.

Despite the recent wealth of literature on the subject of bridge grillages, several aspects of grillage behaviour were still in need of clarification. To cope with the high degree of redundancy of the grillage structure previous methods of analysis were based upon one or more simplifying assumptions, the effect of which was to facilitate computations even at the expense of destroying the accuracy of the solution. It was of the utmost importance that the bridge designer should be aware of the limitations imposed by such simplifications.

The two main questions that arose generally in connexion with the design of a bridge grillage were: in what manner was the load carried by the main beams, and what were the loads on the transverse beams? Although existing theories gave acceptable answers to the first question, that was not true for the second. A search of the literature revealed no Paper containing a theoretical analysis and accurate experimental verification of the shearing forces, bending moments, and torques on the transverse beams. They felt that the methods in which those beams were replaced by a continuous medium (as suggested by the Authors, Massonnet, and others), could not give satisfactory results. For the purpose of making a critical assessment of those existing methods of analysis, Mr Sved and Mr Brooks had devised a method of solution using "relaxation" techniques. The principal advantage of that method over existing solutions was that the only supposition involved was that the beams of the grillage behaved in accordance with the Bernoulli-Euler deflexion theory. The large number of redundancies in the structure rendered anything but a relaxation solution prohibitive. Relaxation solutions published in the past (Southwell) showed very slow convergence and demanded an excessive amount of labour for a reasonably accurate solution. An analysis proved that slow convergence was caused by the fact that locking all but one of the intersection points against deflexion (and/or rotation) gave a structure that was extremely rigid in comparison with the actual one. Because the deflexions increased as the cube of the length of the beam—other quantities remaining constant—a large number of operations was necessary to reach the final values.

In the alternative relaxation method conceived by Mr Sved and Mr Brooks that difficulty was overcome by permitting deflexions and rotations along all the beams affected by an operation, and keeping an account of the linear and angular "gaps" that were caused by those movements between the longitudinal and transverse beams (only a brief outline of the method was given there, since it was intended to publish the whole investigation shortly). By constructing suitable operators, satisfactory convergence in three cycles was obtained. By considering the beams, one at a time, simply supported at their ends and loaded by a unit force at one of the load points, deflexions and slopes at the various intersection points were calculated. Similar calculations gave the effect of bending moments and torques applied at the node points. The values thus obtained were grouped to form the operators mentioned, and were used to calculate the effect of closing a "gap" at one point on the "gaps" at other points. The gradual relaxation of all "gaps" led to the final configuration of the grillage; the results of the calculations showed the deflexion, longitudinal slope, and transverse twist of every beam at every intersection point, and

ıddition, they supplied directly the force, bending moment, and torque transmitted hose points.

'he calculations were verified experimentally by tests on two similar steel bridge ks. Four main beams (1·5 in. × 0·75 in.), of span 15 ft and spaced at 2-ft intervals, e simply supported on rigid abutments. The supports permitted rotation in the two m directions. At intervals of 3 ft the transverse beams were rigidly bolted to the n beams, so that all beam centre-lines were in the same plane. In the first grillage transverse beams were 0·553 in. × 0·276 in., and in the second 1·456 in. × 0·182 in. ıt loads (increments of 35 lb. with a maximum of 105 lb.) were separately applied to ry intersection point. Deflexions relative to a separate reference frame and angular ıtions in the two beam directions were measured at all intersection points and at the ports. Bending moments were checked by thirty-six electric resistance strain (E.R.S.) ges attached to the main beams and twenty-four E.R.S. gauges on the transverse ms.

'he discrepancies between calculated and experimental deflexions and slopes were of order of 2%, whilst in the case of the bending moments the error did not exceed 3%.

'he following main conclusions were drawn from the above investigation:—

(1) Whilst the errors in the deflexions calculated from existing theories were usually small enough to be negligible the agreement on bending moments in the main beams was not as close and depended on such factors, apart from initial simplifying assumptions, as the number of beams in the grillage and the relative stiffness of the transverse and longitudinal beams.

(2) In regard to the moments in the transverse beams, no reasonable correlation could be obtained between experiment and theories previously in existence. That could be attributed, in no small way, to the effect of torsion of the longitudinal beams upon the flexural bending of the transverse beams. Although a great diversity of opinion had hitherto existed in regard to the influence of the torsional rigidity of the beams, the relaxation method mentioned above illustrated that many of those lines of thought were without justification. In the same way, Mr Sved and Mr Brooks felt that the assumption made by the Authors that the twist of the main beams was expressible in terms of a cosine function was questionable. The values obtained by Mr Sved and Mr Brooks for that quantity did not coincide with the function suggested above, and, in point of fact, the form of the main beam twist varied for each beam and also with the type and eccentricity of loading. It was there, in their opinion, that large errors were introduced into the evaluation of both main beam torsion and also flexural bending of the transverse beams. Results taken from the

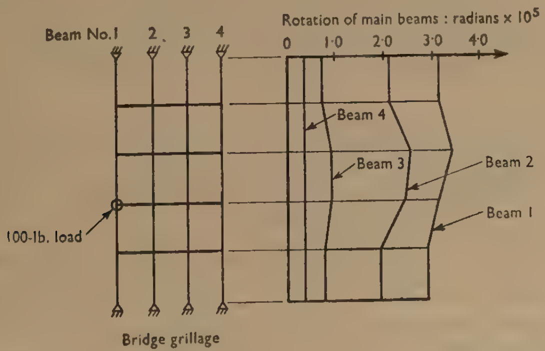

FIG. 12

first of their experimental decks were included as an example. Fig. 12 show the rotation of the main beams under a point load acting upon an edge bea

Dr Z. S. Makowski (Lecturer in Civil Engineering, Imperial College of Science a Technology) said he was very interested in the Paper since he was in charge of a speci post-graduate course on the stress analysis of grids and interconnected bridge girde during which the merits of several methods were discussed. He congratulated t Authors on their clear exposition of a very difficult problem which had exercised t ingenuity of engineers for a very long time. The Paper was extremely useful since dealt with a subject which was now finally attracting an increased amount of attenti from British bridge and constructional engineers. It had always been felt that the was a surprising disproportion between the number of Papers presented on the proble of grids in German or French on the one hand, and in English on the other. Techni literature on the subject was still extremely limited, in spite of some very good resear reports [11] describing the research sponsored by the Cement and Concrete Association the behaviour of various types of reinforced concrete interconnected bridge systems.

The method proposed by the Authors possessed several distinct advantages if co pared with the existing methods. It was general and flexible in that it could be appli to grids supported on two, three, or four sides. It allowed both rotation and twist the members to be taken into account. The proposed method could be applied to a number of girders, whereas existing methods, for example, that due to Pippard Hetenyi,[12] were limited to only three or four main girders. It had a certain superiori over Guyon's [10] and Massonnet's methods, since it could also be applied to bridge syste having outer main girders not necessarily equal to the inner main girders.

The assumption used by the Authors that the transverse girders could be replaced an equivalent continuous medium of constant thickness was a very sound one—it h been used for the first time in the bridge analysis by Pippard and had proved to be tr experimentally even for a very small number of those girders. In Dr Makowski's opini the use of the harmonic analysis was an additional important contribution which grea simplified the solution of the differential equations, which otherwise would become ve tedious.

However, some practical engineers would perhaps be disappointed in finding (as Dr Makowski had done, going through several practical examples of grid analys that the proposed method was not a very quick one. In cases in which the effect of t torsional rigidity was included in the analysis, the Authors' method tended to beco rather time-consuming for the following reasons:—

(1) The convergence of the infinite series, as used in the method, was often ve slow. The Authors pointed that out on p. 947, but Dr Makowski wondered many readers realized the practical importance of it. The first step in t method was the determination of the deflexions (which were usually of lit interest to a designer) from which bending moments and shearing forces co be obtained.

Dr Makowski found that the amount of work required for the determinati of deflexions compared quite favourably with similar existing methods si usually only one harmonic needed to be considered. Unfortunately he fou also, in all the considered cases, that, owing to the slow convergence of t series, at least three harmonics had to be taken for bending moments and oft five harmonics for shearing forces.

(2) The suggested method could be applied directly when only *one* longitudinal w loaded. For loads acting on several longitudinals, whose free deflexions w not equal, the analysis had to be repeated several times, i.e., for each load longitudinal separately, and the final displacements obtained then by t principle of superposition. It was true that for a given grid the same gene equations were used for any loading case and solution was obtained by t substitution of the appropriate number, but nevertheless the amount of pur

numerical work in such cases was increased considerably. The suggested method might, however, prove very useful for the determination of influence lines, for which single unit loads were considered in the analysis.

)r Makowski agreed that one of the advantages of the Authors' method was that it ld take into account the influence of the torsional rigidity of various members. He nd, however, that the Authors tended to over-emphasize the influence of that factor . it could be easily noted that all the examples discussed in the Paper could hardly be ted as typical ones, owing to the unusually high ratio of the torsional to bending ness of the members.

ligidity of connexions influenced to a certain extent the stress distribution in grids, even for typical reinforced concrete grids its effect was much smaller than implied by numerical examples given in the Paper.

Ie agreed with the Authors that if torsional stiffness was negligible their method uced greatly in length and became simpler and extremely useful for grids supported ng three or four sides.

'or steel interconnected bridge systems, supported on two sides only, Dr Makowski's sonal experience was that Leonhardt's [9] method was much quicker, giving results ch compared very favourably with those obtained by the more precise (and much re complicated) methods. Leonhardt's method had the additional advantage that ocused its attention on the bending moment and shearing forces, whose diagrams e obtained first, from which deflexions could also be deduced if required.

)r Makowski was also interested in the experimental work described on p. 950, and ecially in the conclusion reached by the Authors that in the interconnected bridge lers the position of the cross-girders along the span was not critical. The Authors arked that as one moved the cross-girder from mid-span towards the supports, the nsmission by shear decreased as the transmission by torsion increased. That statent was correct and the Authors' conclusion was also valid, but only in the specific considered of a grid having an exceptionally high ratio of torsional to flexural stiffness. vas, however, not generally true, especially for grids of negligible torsional rigidity. It ld be shown that in those cases a cross-girder was most effective only if placed in the -span of the bridge. Dr Makowski had discussed the problem of the effectiveness of position of the cross-girders elsewhere [13] and would like to point out that a comparison of the diagrams of the maximum bending moments for a series of bridges, having same number of main girders and under the action of the same live load but internected by a variable number of cross-girders, proved that in fact one cross-girder ced at the mid-span (i.e., at $L/2$) was equivalent to two cross-girders placed at $L/3$ $2L/3$. Similarly, three cross-girders at $L/4$, $L/2$, and $3L/4$ produced a diagram of kimum bending moment almost identical with that for four cross-girders placed at , $2L/5$, $3L/5$, and $4L/5$.

t was therefore a good practice from the theoretical point of view to have an odd nber of cross-girders, usually one or three for small-span bridges and perhaps five for er spans. In each case one of the cross-girders should be placed at the mid-span, re it was *most effective* in the lateral load distribution.

eferring to the experimental analysis of grid frameworks using small-scale models, Makowski said the experimental work described by the Authors was of necessity of ted character and its sole aim was to verify the legitimacy of the assumptions used heir analytical method. Small-scale models of grids could be used also to obtain rmation about the behaviour of the prototype structure and generally the data from lels was reliable and could be used with confidence for design purposes.

t the Imperial College simple models of grid frameworks were used to determine ctly the influence surfaces for the reactions between main and cross-girders, or the ence surfaces of bending moments and shearing forces for any section of the main cross-girders. The agreement between experimental and analytical values was rally very good.

he experimental approach was very efficient for complicated grids having a large

number of members of different dimensions and spacing, for skew-bridges, and othe unsymmetrical layouts for which the mathematical analysis became tedious. The co of the model was usually small and the amount of time required to take the readings w only a small portion of the time required for the analysis even if that was simplified b the use of Tables. The advantages of the experimental approach increased for spa grids, whose analytical treatment was very complicated.

The Authors, in reply, stated that Mr Kendrick had contributed the results of a seri of calculations by relaxation methods which he claimed represented an exact solution f the grid in Fig. 3a; Mr Kendrick's results for assumption (*c*) (i.e., Pippard's assumption agreed with those given by the Authors' theory. His results for assumption (*a*) (ze torsion) also agreed with the Authors' solution as shown in Table 11.

TABLE 11.—INTERSECTION DISPLACEMENTS FOR GRID OF FIG. 3a: ZERO TORSIO

(All values in inches)

Point	Kendrick's assumption (*a*)	Hendry and Jaeger	Experiment
11	0·118	0·120	0·123
12	0·206	0·214	0·208
13	0·0887	0·089	0·090
21	0·0303	0·031	0·0316
22	0·0588	0·059	0·0612
23	0·0285	0·030	0·0296
31	0·00575	0·005	0·0050
32	0·0105	0·010	0·0108
33	0·00476	0·005	0·006
41	0·0181	0·018	0·019
42	0·0365	0·038	0·038
43	0·0184	0·018	0·019

In Mr Kendrick's case (*b*) (zero torsional stiffness of the transversals, finite torsio stiffness of the longitudinals) his results were at variance with the Authors' theoretical a experimental results; Mr Kendrick had put forward a few suggestions to account for d crepancies which he believed existed in the Authors' work. He suggested that frictio effects might have been present in the experimental frame but the Authors had check experimental deflexions and affirmed that such effects did not exist; furthermore all experimental deflexions were correct to within one or two thousandths of an inch. Kendrick had questioned the EI-values of the members of the test frame. That difficu arose from a misprint in Fig. 3a which gave the applied load as 1 lb. instead of 2 lb. T actual EI-values were thus double the values derived by Mr Kendrick from the numeri example constants. The error was regretted but was obvious because all the flexu rigidities he quoted were about half the nominal values.

The experimental results being correct for the frame tested, it followed that Mr K drick's calculation was not "exact" in relation to the actual frame tested and made allowance for torsion in the transverse direction, increased effective torsional stiffnes the longitudinals resulting from prevention of warping at the points of attachment of cross-girders, and the finite size of the members. That frame represented an extreme c so far as the Authors' theory was concerned since it did not conform at all closely assumptions made in the theory; it had only three transversals, which was far from be a continuous medium, and in it the variation of θ along the length of the longitudinals m be a saw-tooth function, represented only roughly by $(c + \lambda \cos \pi x/L)$. It was, theref remarkable that agreement between theoretical and experimental results should be so cl

wever, it could hardly be considered fortuitous that agreement within design limits was ained in view of the similar agreement obtained in the experiments shown in Table 1, s 3b, 6, 8, and 10 and in cases reported elsewhere.[18] The Authors' aim was to produce ractical method of analysis which made adequate allowance for torsional effects. They not claim that the method was "exact"; merely that it could be relied upon to give ults of sufficient accuracy for practical purposes. If, however, theoretical refinements re desired the theory was readily modified. In the case discussed by Mr Kendrick the umption of the form $(c + \lambda \cos \pi x/L)$ for the variation of θ along the longitudinals led some exaggeration of torsional stiffness; a better approximation for the θ function would $(c + \lambda \cos 2\pi x/L)$. That assumption together with allowance for transverse torsion and te size of members resulted in a solution of the same order of accuracy as the original ory.

Mr Kendrick's proposal to use a differential equation condition for torsion was theoretic- impeccable but to put into effect was another matter and the Authors could not see at useful purpose would be served by so doing. Similarly, the Authors were un- pressed by Mr Kendrick's lament for the lost harmonics of λ since those evidently had no ctical effect on the solution.

The purpose of the Paper was to present a method of analysis which allowed for torsion and when necessary; it was not feasible within the limits of the Paper to undertake a ailed discussion of the errors involved in making simplifying assumptions. The Authors l, however, by no means overlooked the simplification of omitting torsional rigidity Mr Kendrick asserted* but they did not agree with the suggestion that a satisfactory ution would be obtained in most practical structures by neglecting torsion; that would y be true when the parameters α and β were both small, e.g., in a bridge with members steel I section.

t was perhaps not sufficiently emphasized in the Paper that the Authors' method was entially a process of distributing harmonics whether they were harmonics of the bending ment or the deflexion curve. It was obvious that only one or two harmonics would be uired to obtain an accurate definition of the deflexion curves of the longitudinals. That ld give a series with only one or two terms; it was equally obvious that in general it ld not be sufficient to differentiate the series twice to give the corresponding bending. ment curve since the latter would require more harmonics for accurate representation. that meant was that a larger number of harmonics of the free bending-moment diagram ld have to be distributed, as shown on p. 955. It seemed that Mr Kendrick had not reciated that point.

Ir Chaudhuri and Dr Makowski had both made comparisons between the Authors' hod and various other existing theories. That was of considerable interest and im- tance and had not been attempted in the Paper which was, as mentioned, intended ely as an exposition of the principles of the Authors' method. The development of the hod into a really satisfactory design tool had now been achieved, solutions were avail- for a wide range of structures in the form of graphs or formulae of distribution co- ients.[14–18] With those at hand it was only necessary to carry out the sort of numerical ution shown in the Paper in special cases, e.g. when the spacing of the main girders was gular.

Ir Chaudhuri's worked example was very interesting and a comparative study of the lts was instructive. The Authors did not agree with his conclusion that a torsionless lysis would be adequate in the case he had taken. Comparison of Tables 3 and 6 wed a difference of more than 25% in the maximum deflexions of the loaded girder. y agreed, however, that neglect of torsion was correct in steel bridge girders and should dopted for concrete structures and wherever there was doubt on their continuity.

Ir Chaudhuri and Dr Makowski also referred to the question of positioning of cross- ers. The Authors had investigated that analytically for torsionless structures where

* See p. 956 of the Paper and references 14–17 on p. 864.

6

it was of the greatest importance; they found that the following expression gave th effective number of cross-girders for use in calculating the parameter α in the analysis of torsionless frame:

$$n_{\mathrm{eff}} = \left(n + \Sigma \cos \frac{2\pi e}{L}\right)$$

where n denoted the actual number of cross-girders and e_1, e_2 . . . the distances of thos members from mid-span. One result readily seen from that formula was that if a give transverse flexural rigidity was concentrated at mid-span the effective n would be twic that when the same EI was spread over the whole span.

Messrs Sved and Brooks had developed a useful relaxation technique for solution of gri frame problems which should prove valuable for special investigations. Unfortunatel relaxation methods provided solutions to only one case at a time and it was difficult by suc methods to obtain a picture of the behaviour of grid structures generally and, in particula to discover what were the significant parameters governing that behaviour. As previousl observed, it had been found possible to obtain general solutions for a wide variety of prob lems by the Authors' method and even to extend its application to slabs and cellular plate structures.[18]

The treatment of transverse moments had been dealt with in greater detail by the Autho elsewhere[15, 16] and it was believed that those moments could be obtained with reasonab accuracy. The estimation of transverse moments in an actual structure was, howeve attended by considerable difficulty and always likely to be far less accurate than th calculation of longitudinal moments no matter what method of analysis was employed.

Referring to Messrs Sved and Brooks's experimental results shown in Fig. 12, it might b observed that the members used for that grid possessed relatively high torsional stiffne ($\beta = 4{\cdot}1$ for the frame) and the major part of the rotation of each beam was of the "rigi body" type. The error involved in using the function $\lambda \cos \pi x/L$ for the variation of θ alon the beams would at any point be only a small fraction of the total rotation; indeed tha frame could with good accuracy be analysed on the basis of infinite torsional stiffness the longitudinals.

There was a certain irregularity in the rotations obtained by experiment, perhaps diffi cult to avoid, having regard to the difficulty of measuring minute rotations and of avoidin secondary effects owing to eccentricities of beam connexions and the like, but a compariso between experimental and theoretical rotations and an analysis of the effect of small erro in θ on the transverse moments would certainly be instructive.

The Authors agreed with Dr Makowski in recommending the use of small-scale mode for the analysis of analytically difficult cases and suggested that their approach throug the use of distribution coefficients for harmonics could be profitably applied in conjunctio with model tests, i.e., the model tests might be directed towards the determination distribution coefficients for harmonics which would subsequently be applicable to an loading condition.

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CORRESPONDENCE

on Papers published in Proceedings, Part III, April, 1956

Paper No. 6077

A design chart for the economic section for prestressed concrete beams [†]

by

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Correspondence

Mr N. J. Cochrane (Senior Engineer, Sir William Halcrow & Partners) suggested tha the Author might have carried his analysis somewhat further. As it stood, the Paper se out a design chart, but not necessarily a truly economic one, since the Author wa apparently not concerned with costs.

In 1944, Mr Cochrane had suggested[6] a method for dealing with the costs of alternativ simple shapes. He thought that the Author's shapes were also fairly simple and h suggested that they might be amenable to a true economic analysis also. A practisin designer had to bear in mind that stress economy and cost economy might be quit different things and he considered that the Author's interesting series of Papers could b rounded off by an analysis of the type suggested above.

The Author, in reply, was pleased that Mr Cochrane had expressed interest in hi series of Papers since a fourth was in preparation dealing with beams forming a deck an with composite construction.

Mr Cochrane had suggested that "the Author was apparently not concerned with cost but reference to the first Paper of the series[*] would show that that was not so.

It had been pointed out there that a true valuation of economy had to take accoun of the effect of an increase in depth on the cost of a project as a whole, as well as c the two principal components involved in the cost of the beam, the cable and th volume of concrete.

The present Paper gave all three quantities separately, in terms of the maximum stres so that the practising designer could readily assess the comparison between "stre economy" and "true economy" as Mr Cochrane desired, with reference to the actu economy for any given project.

Although the present chart illustrated only one ratio of upper flange width to bea depth ($\frac{1}{2}$) the effect of varying that ratio had been discussed on p. 193 of the present Pape

* R. G. Robertson, "Prestressed concrete beams: the economical shape of section" Proc. Instn Civ. Engrs, Pt III, vol. 3, p. 242 (Apr. 1954).

† Proc. Instn Civ. Engrs, Pt III, vol. 5, p. 184 (Apr. 1956).

6 N. J. Cochrane, "The economics of reinforced concrete sections". J. Instn Civ. Engr vol. 23, p. 155 (Jan. 1945).

nd all possible ratios were shown on the chart given in the first Paper which could be ferred to for the effect of a different ratio (b/d).

Comparison of designs for the same stress but different depths, and for the same depth ut different stresses had been made in the first Paper; that might be the type of cost nalysis requested by Mr Cochrane: in view of that the two assumptions made there for e cost comparison might be amplified.

First, the same basic cost of concrete was used with different stresses since a high-grade oncrete was required to avoid creep loss irrespective of the stress used; that resulted in a wer total cost when allowing a higher stress.

Secondly, the ratio of the cost of the cable to the cost of the completed beam was ssumed to be 70% in the basic design used for comparison but altering that to 50% made o difference to the conclusions.

Those conclusions were different in the case of beams forming a deck and the Author as glad that Mr Cochrane had raised the matter which would be attended to on the lines e suggested in the proposed Paper on composite construction.

Paper No. 6085

The effect of uplift on gravity-dam profiles †

by

Eric Hugh Brown, Ph.D., B.Sc.(Eng.), D.I.C., A.M.I.C.E.

Correspondence

Dr O. C. Zienkiewicz (Lecturer, Department of Engineering, University of Edin- rgh) stated that the validity of the conclusions and the profile formulae derived, useful they would doubtless be for economical design, hinged on the validity of one basic sumption. That was the linear distribution of the vertical component of "material" ess on horizontal sections under all conditions of uplift pressures.

That linearity, approximately true with zero uplift, appeared, if Brahtz's conclusions re generally true, to be realized only if the uplift pressures were also linear. It thus lowed that if Brahtz's conclusions could be extrapolated beyond the limits of their ict applicability, the results were of little use; furthermore, any benefits of internal inage were nullified in all gravity dams.

That vexing problem had intrigued Mr Zienkiewicz for some time and had prompted ne detailed analytical work which gave an answer of practical value. A Paper on the eme had recently been published.[7] It was proposed to quote some of the results and the proach used.

To make a full stress analysis with uplift amenable to a mathematical approach, a nplified dam (a triangular section of large height) was considered and the uplift variation s assumed to have the same shape at all similar sections.

It was well known that the profile chosen exhibited an exactly linear stress distribu- n with no uplift and approximated elastically to the portions of the dam not affected the foundation contact.

Proc. Instn Civ. Engrs, Pt III, vol. 5, p. 196 (Apr. 1956).

O. C. Zienkiewicz, "The effect of pore pressures on stresses in gravity dams". Pow. Divn, No. Po. 4, Proc. Am. Soc. C.E. (Aug. 1956).

The basis for the analysis was provided by the general equations of plane stress elasticity in which the effect of uplift or internal pore pressure was introduced by body forces proportional to the pressure gradients.

Defining the three stress components in terms of a stress function ϕ as followed:

$$\left.\begin{array}{ll}\text{Stress in X direction:} & \sigma_x = \dfrac{\partial^2\phi}{\partial y^2} + p\beta - w_s y \\ \text{Stress in Y direction:} & \sigma_y = \dfrac{\partial^2\phi}{\partial x^2} + p\beta - w_s y \\ \text{Shear stress:} & \tau_{xy} = -\dfrac{\partial^2\phi}{\partial x \partial y}\end{array}\right\} \quad \ldots \quad (31)$$

where y was measured downwards, w_s was the unit weight of the saturated material, β an area coefficient, and p the uplift pressure, it followed from elastic compatability, that:

$$\nabla^2(\nabla^2\phi + Kp) = 0 \quad \ldots \quad (32)^*$$

where $$\nabla^2 = \frac{\partial^2}{\partial x^2} + \frac{\partial^2}{\partial y^2}$$

and $$K = \frac{1-2\nu}{1-\nu}\beta \quad (\nu = \text{Poisson's ratio})$$

Brahtz's conclusions, mentioned on p. 198 of the Author's Paper, were based on the particular distribution of p where $\nabla^2 p = 0$ reducing the equations to a simple form. Solutions of cases where that relation was not satisfied were obviously of prime interest

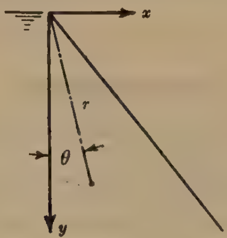

FIG. 18

For a very high triangular profile shown in Fig. 18, the uplift pressure in general coul be assumed to be of the type:

$$p = r\Psi \quad \ldots \quad (33)$$

where Ψ was a function of angle θ only. With that stipulation it was found that th stress function was of the form:

$$\phi = r^3 f \quad \ldots \quad (34)$$

where f again was a function of the angle only. Substitution into equation (2) resulte in a simple ordinary differential equation:

$$\frac{d^4 f}{d\theta^4} + 10\frac{d^2 f}{d\theta^2} + 9f = -K\left(\Psi + \frac{d^2\Psi}{d\theta^2}\right) \quad \ldots \quad (35)$$

which could be solved for any assumed distribution of pressure, i.e., a known function Ψ

One example followed, shown in Fig. 19. In that case, a line of drainage was assume to exist along a radial plane reducing the pore pressure from the full hydrostatic head a the upstream face to zero at that line. Extension of Brahtz's hypothesis on similar lin

those used in deriving Fig. 5 resulted in a "material" stress distribution shown in g. 19b. The correct elastic solution derived from equation (35) resulted in the dis- ibutions of material stress shown in Fig. 19c. Those results were interesting because ey depended appreciably on the values of Poisson's ratio assumed.

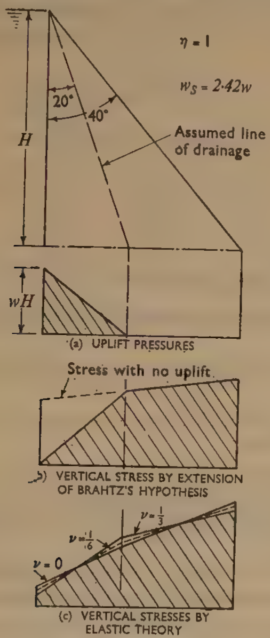

Fig. 19

t was striking, however, that the departure of the material stresses distribution from arity, in the example chosen, was fairly small for usual values of the Poisson's ratio. nerous other cases of uplift distribution had been investigated on those lines, with same general result. That was indeed fortunate validating as it did, at least approxi- ely, the basic assumption on which the results of the Author's work were based. It hoped that it would renew the confidence many engineers placed in suitable drainage ces which reduced the resultant uplift force without necessarily decreasing the maxi- n value of the pore pressure.

n conclusion, it should be mentioned that Brahtz's hypothesis was already extended ond the strict limits of its applicability even in the apparently simple case given in 3–5. The Author's conclusion that the drainage provided by horizontal galleries of no avail in reducing the stresses was, therefore, founded on false premises and ired careful examination.

r James Park (Research Engineer, W. S. Atkins and Partners) observed that when loying Brahtz's conclusions on the effect of pore pressure on stresses in porous bodies, as always necessary to examine carefully whether the problem fell strictly within the ds of applicability of Brahtz's results. In so doing it was useful to examine the se terms in which Brahtz defined his main result*:

Up to that point, the analysis followed closely the method outlined in Brahtz's nal work. (*See* reference 5 of Paper, p. 206.)

"In order to obtain the contact stresses [pore pressure acting] the mean stresse determined by the boundary and body forces in the absence of pore pressure must b decreased by an amount βp if internal liquid pressure is also present",

where β denoted the area factor and p the pore pressure.

The proof of that statement was fairly straightforward. The stress components, σ_x σ_y, and τ_{xy} were expressed in terms of a stress function ϕ thus:

$$\left.\begin{aligned} \sigma_x &= \frac{\partial^2\phi}{\partial x^2} - w_s y + \beta p \\ \sigma_y &= \frac{\partial^2\phi}{\partial y^2} - w_s y + \beta p \\ \tau_{xy} &= -\frac{\partial^2\phi}{\partial x \partial y} \end{aligned}\right\} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (36)$$

where w_s was the saturated weight of the material. The term βp was omitted for a nor porous body.

By considering the condition for compatibility of strain, and the boundary condition it was easily shown that:—

(1) The general form of the stress function ϕ was identical for porous and non-porou bodies.

(2) The arbitrary constants in the general expression for ϕ were identical for porou and non-porous bodies.

Thus, since ϕ was not dependent in any way on the porosity of the material, Brahtz conclusion follows from equations (36), from which it was seen that the normal stress in porous and non-porous bodies differed only by an amount βp. The shear stress τ_{xy} wa not affected by pore pressure.

It was an essential feature of Brahtz's conclusion that the pressure function p shou obey the Laplace equation throughout the region considered.

In the example considered by the Author under the heading "The effect of drains on t stress distribution", the presence of the longitudinal gallery created a problem beyond t scope of Brahtz's result, for the transverse section of the dam was, in terms of the theo of elasticity, a region of multiple connexion; at least it might be considered such if t gallery was sufficiently large compared with the area of the section. Otherwise, if t gallery was treated as a point drain, the pressure function was not harmonic at that poi and Brahtz's conclusion was therefore immediately inapplicable. Assuming, however, th the gallery was of reasonable size, the section must be considered as a region of multip connexion, in the "solid" part of which the pressure function p obeyed the Lapla equation. It was a general rule that in such a region the boundary and body forces we insufficient to determine the stress distribution, and displacements must also be co sidered, usually imposing the condition that they must be single-valued functions on It was debatable whether Brahtz was aware of that limitation when he worded his co clusion specifying stresses "determined by the boundary and body forces", but the nec sity for imposing additional conditions made Brahtz's conclusion inapplicable in region of multiple connexion, for the proof of the conclusion was based on the calcu tion of stress for a simply-connected region only.

Cylinder problem

A simple example illustrated the type of discrepancy involved, without going into t elaborate calculations required to analyse the section of a gravity dam. In the tw dimensional problem of a section of a porous cylinder, of internal radius a and exter radius b, subjected to an internal water pressure p_0 with a resulting radial percolation water, if the cylinder were non-porous, the radial and tangential stress components wo be, respectively:

$$\left.\begin{aligned} \sigma_r &= \frac{p_0 a^2}{(b^2 - a^2)}\left[1 - \frac{b^2}{r^2}\right] \\ \sigma_\theta &= \frac{p_0 a^2}{(b^2 - a^2)}\left[1 + \frac{b^2}{r^2}\right] \end{aligned}\right\} \quad . \; . \; . \; . \; . \; . \; . \quad (37)[8]$$

n a porous cylinder, assuming that radial flow obeyed Darcy's law so that $\nabla^2 p = 0$, e pressure distribution was given by:

$$p = p_0 \frac{\log b/r}{\log b/a}$$

here log denoted logarithms to the base e.

The stress components obtained by applying Brahtz's result were then:

$$\left.\begin{aligned} \sigma_r &= \frac{p_0 a^2}{(b^2 - a^2)}\left[1 - \frac{b^2}{r^2}\right] + \beta p_0 \frac{\log b/r}{\log b/a} \\ \sigma_\theta &= \frac{p_0 a^2}{(b^2 - a^2)}\left[1 + \frac{b^2}{r^2}\right] + \beta p_0 \frac{\log b/r}{\log b/a} \end{aligned}\right\} \quad . \; . \; . \; . \; . \quad (38)$$

If, however, the problem was worked out from first principles, the equilibrium equation ntained the body-force term $-\beta\dfrac{dp}{dr}$ and the true stress components were found to be:

$$\left.\begin{aligned} &= \frac{p_0 a^2}{(b^2 - a^2)}\left[1 - \frac{b^2}{r^2}\right]\left[\frac{\beta}{2(1-\nu)} + 1 - \beta\right] + \frac{\beta p_0}{2(1-\nu)} \frac{\log\frac{b}{r}}{\log\frac{b}{a}} \\ &= \frac{p_0 a^2}{(b^2 - a^2)}\left[1 + \frac{b^2}{r^2}\right]\left[\frac{\beta}{2(1-\nu)} + 1 - \beta\right] + \frac{\beta p_0}{2(1-\nu)} \frac{\log\frac{b}{r}}{\log\frac{b}{a}} + \frac{\beta p_0}{2\log\frac{b}{a}} \frac{(1-2\nu)}{(1-\nu)} \end{aligned}\right\} \quad (39)$$

here ν denoted Poisson's ratio.

To determine how the discrepancy arose between equations (38) and (39), Brahtz's oof must be re-examined with regard to multiply-connected bodies.

For a non-porous section with axial symmetry, the stress components could be expressed terms of a stress function ϕ such that:

$$\left.\begin{aligned} \sigma_r &= \frac{1}{r} \cdot \frac{d\phi}{dr} \\ \sigma_\theta &= \frac{d^2\phi}{d\theta^2} \end{aligned}\right\} \quad . \; . \; . \; . \; . \; . \; . \; . \quad (40)$$

pressions (40) automatically satisfied the equation of equilibrium:

$$\frac{d\sigma_r}{dr} + \frac{\sigma_r - \sigma_\theta}{r} = 0$$

hen there was no body force the condition of compatibility was:

$$\nabla^2(\sigma_r + \sigma_\theta) = 0$$

d substituting expressions (40) resulted in:

$$\nabla^4 \phi = 0 \quad . \; . \; . \; . \; . \; . \; . \; . \; . \quad (41)$$

For a porous section the stress components became:

$$\left.\begin{aligned} \sigma_r &= \frac{1}{r}\frac{d\phi}{dr} + \beta p \\ \sigma_\theta &= \frac{d^2\phi}{dr^2} + \beta p \end{aligned}\right\} \quad . \; . \; . \; . \; . \; . \; . \quad (42)$$

[8] S. Timoshenko and J. N. Goodier, "Theory of elasticity." 2nd edn. McGraw-Hill, w York, 1951.

The equilibrium equation, which again was automatically satisfied was:

$$\frac{d\sigma_r}{dr} + \frac{\sigma_r - \sigma_\theta}{r} - \beta r \frac{dp}{dr} = 0$$

The condition of compatibility became:

$$\nabla^2(\sigma_r + \sigma_\theta) = \frac{\beta}{(1-\nu)} \nabla^2 p,$$

and since $\nabla^2 p = 0$, substituting expressions (42):

$$\nabla^4 \phi = 0 \qquad \ldots \ldots \ldots \qquad (43)$$

Thus the general form of the stress function, as seen from equations (41) and (43), was the same for porous and non-porous bodies, as was found in part (1) above of Brahtz's proof for simply-connected bodies. In polar co-ordinates the solution of (43) had the form :

$$\phi = A \log r + Br^2 \log r + Cr^2 + D$$

and the stress components, from (40) and (42) were:

$$\left.\begin{aligned} \sigma_r &= \frac{A}{r^2} + B(3 + 2\log r) + 2C + \beta p \\ \sigma_\theta &= -\frac{A}{r^2} + B(3 + 2\log r) + 2C + \beta p \end{aligned}\right\} \qquad \ldots \qquad (44)$$

the last term in each expression being omitted in the case of non-porous bodies.

Consider now the arbitrary constants A, B, and C. The value of D did not need to be considered since it did not appear in the stress equations. Only two boundary conditions existed, namely the conditions of normal stress at the inner and outer boundaries. Therefore to evaluate the three constants it was necessary to consider displacements, imposing the condition that they must be single-valued functions of r only in an axially symmetrical system.

Displacements

The components of radial, tangential, and shear strain (ϵ_r, ϵ_θ, and $\gamma_{r\theta}$ respectively) could be expressed in terms of the radial and tangential displacements, u and v, as followed:

$$\epsilon_r = \frac{\partial u}{\partial r} \qquad \ldots \ldots \ldots \qquad (45a)$$

$$\epsilon_\theta = \frac{u}{r} + \frac{1}{r}\frac{\partial v}{\partial \theta} \qquad \ldots \ldots \ldots \qquad (45b)$$

$$\gamma_{r\theta} = \frac{1}{r}\frac{\partial u}{\partial \theta} + \frac{\partial v}{\partial r} - \frac{v}{r} \qquad \ldots \ldots \ldots \qquad (45c)$$

The values of u and v were now required.

Using Hooke's Law, the strain components could also be expressed in terms of the stress components:

$$\epsilon_r = \frac{1}{E}[(1-\nu^2)\sigma_\theta - \nu(1+\nu)\sigma_r] \qquad \ldots \ldots \qquad (46a)$$

$$\epsilon_\theta = \frac{1}{E}[(1-\nu^2)\sigma_\theta - \nu(1+\nu)\sigma_r] \qquad \ldots \ldots \qquad (46b)$$

$$\gamma_{r\theta} = \frac{2(1+\nu)}{E} \tau_{r\theta} \qquad \ldots \ldots \ldots \qquad (46c)$$

rom equations (45a) and (46a), substituting the expression (44):

$$= \frac{1}{E}\left[\frac{A}{r^2}(1+\nu) + B(1-3\nu-4\nu^2) + 2B\log r(1-\nu-2\nu^2) + 2C(1-\nu-2\nu^2) + \beta(1-\nu-2\nu^2)p\right]$$

l integrating with respect to r:

$$= \frac{1}{E}\left[-\frac{A}{r}(1+\nu) - Br(1+\nu) + 2Br\log r\,(1-\nu-2\nu^2) + 2Cr(1-\nu-2\nu^2) + \beta(1-\nu-2\nu^2)\int pdr\right] + f(\theta) \quad . \; . \; . \; . \; . \; . \; . \; . \; . \; . \quad (47)$$

ere $f(\theta)$ was a function of θ only, and $\int pdr$ represented the general integral only, the itrary constant of integration being included in $f(\theta)$.

rom (45b) and (46b):

$$\frac{\partial v}{\partial \theta} = r\epsilon_\theta - u$$

$$= \frac{r}{E}\left[(1-\nu^2)\sigma_\theta - \nu(1+\nu)\sigma_r\right] - u$$

l substituting from equations (44) and (47):

$$= \frac{1}{E}[4Br(1-\nu^2) + \beta(1-\nu-2\nu^2)\{pr - \int pdr\}] - f(\theta)$$

egrating:

$$\frac{1}{E}[4Br(1-\nu^2)\theta + \beta(1-\nu-2\nu^2)\{pr - \int pdr\}\theta] - \int f(\theta)d\theta + f(r) \quad . \; . \; . \quad (48)$$

re $f(r)$ was a function of r only.

or a system of symmetrical stress distribution the shear stress ($\tau_{r\theta}$) was zero, therefore was also zero. Thus, substituting (47) and (48) in equation (45c):

$$\frac{1}{r}\frac{df(\theta)}{d\theta} + \frac{df(r)}{dr} + \frac{1}{r}\int f(\theta)d\theta - \frac{1}{r}f(r) = 0$$

n which, by separating functions of r and θ:

$$f(r) = F_1 r + F_2 \text{: } f(\theta) = F_3\sin\theta + F_4\cos\theta \quad . \; . \; . \; . \quad (49)$$

F_2, F_3, F_4, being constants.

ubstituting expressions (49) in (47) and (48):

$$\left[-\frac{A}{r}(1+\nu) - Br(1+\nu) + 2Br\log r(1-\nu-2\nu^2) + 2Cr(1-\nu-2\nu^2) + \beta(1-\nu-2\nu^2)\int pdr\right] + F_3\sin\theta + F_4\cos\theta \quad . \; . \; . \quad (50)$$

$$\frac{4Br(1-\nu^2)}{E}\theta + \frac{\beta}{E}(1-\nu-2\nu^2)[pr - \int pdr]\theta + F_3\cos\theta - F_4\sin\theta + F_1 r + F_2 \quad . \; . \; . \quad (51)$$

h were the required results.

onsidering now the section of a cylinder, from symmetry, displacements could occur in radial planes. v must therefore be zero for all values of θ.

followed from equation (51) that:

$$\frac{4Br(1-\nu^2)}{E} + \frac{\beta}{E}(1-\nu-2\nu^2)[pr - \int pdr] = 0$$

$$F_1 = 0$$
$$F_2 = 0$$
$$F_3 = 0$$
$$F_4 = 0$$

r a non-porous body, the pore-pressure terms involving p must vanish and so $B = 0$.

However, where pore pressure existed:

$$B = -\frac{\beta}{4}\frac{(1-2\nu)}{(1-\nu)}\frac{1}{r}\left[pr - \int p\,dr\right]$$

$$= -\frac{\beta}{4}\frac{(1-2\nu)}{(1-\nu)}\frac{1}{r}\left[pr - p_0\int\frac{\log\frac{b}{r}}{\log\frac{b}{a}}\,dr\right]$$

$$= \beta\frac{p_0(1-2\nu)}{4\log\frac{b}{a}(1-\nu)}$$

The two remaining constants A and C could be determined from the boundary conditions. Without going farther it had been shown that the arbitrary constants in the general expression for the stress function were not identical for porous and non-porous multiply-connected bodies. The difference arose when imposing the condition for single valued displacements, when an additional term, caused by the body force, appeared in the expressions for displacements (50 and 51). Such a term would always appear. In some particular case its value might be zero, depending on the pressure distribution: and only in such a case would Brahtz's result be applicable.

In general, however, the result was inapplicable in multiply-connected regions.

That conclusion did not seriously affect the main conclusions of the Author's Paper but it was one which was not generally recognized and which sometimes led to an erroneous interpretation of Brahtz's results.

Mr J. L. Serafim (Research Engineer, Head of Dams Section, Laboratório Nacional de Engenharia Civil, Lisbon) stated that probably, in design of dams, there was no problem subject to such extensive debate as that of uplift. In recent discussions[9, 10] the main results of the experimental findings of the area factor carried out at the Laboratório Nacional de Engenharia Civil in Lisbon were given. Such results proved that the value of the area factor for deformations in the elastic range differed from the one in rupture. It also depended on the dryness of the concrete, the percolating fluid, the characteristics of the concrete, the value of the pore pressure, etc. Values from about 0·4 to 1·0 had been obtained.

The Author did not discuss that problem and referred to Levy's classical treatment in which nothing was stated about the "proportion of area over which uplift was assumed to act" since Levy postulated the existence of a crack extending from upstream to downstream in the dam. Surely that assumption was equivalent to an area factor of 100% which was now generally accepted, but it did not conform to the treatment of material stresses, i.e., effective stresses in a porous body, as made in the Paper.

The Author wondered "if Brahtz's conclusion for a two-dimensional system is more generally valid . . .", but it has already been proved that it was not.[11] That meant, in a three-dimensional state of stress (and of percolation) the uplift stresses could not be obtained by deducting the value of the pore pressure from the total stresses assuming that the body was impermeable. In fact, even if the percolation of the water through the dam was in a steady state ($\nabla^2 p = 0$), a state of hydrostatic (pantatenic) stresses equal to p was not possible. That meant it did not satisfy the compatibility conditions of the theory of elasticity.

The Author also stated that there was no published guidance on the effect of uplift on the distribution of the "material-contact stresses" in the case of vertical drains and of a region of denser concrete at the upstream face. However, both cases could be treated and were presented in a publication on the various aspects of the uplift problem[11].

In view of the ideas following the statement that "Uplift pressure would thus be

[9] References 9–11 are given on p. 875.

come phenomenon . . .", it was suggested that the Author should study in detail the e aspects and other physical conditions of the development of the uplift in concrete.

The Author, in reply, expressed his appreciation of the various comments and agreed h Mr Serafim that the difficulty in reaching a generally acceptable account of uplift in the number of aspects of the physical problem which were uncertain.

It had been the Author's intention to present a clear-cut and, it was hoped, useful ution to what he had termed the *structural* problem, on certain stated assumptions, and m the solution to see the effect of certain variables. The solution involved no difficulties; vas as valid as the assumptions on which it was based. The most questionable of those umptions was the customary one of linearity of vertical stress in the concrete, and the thor had deliberately drawn attention to it, for that one assumption evaded many of the troversial difficulties of the uplift problem. He was grateful to Dr Zienkiewicz and Park for their arguments in support of linearity, which encouraged the hope that when was understood the simple linear theory might be found to give a good approximation. One of the more disturbing doubts about the assumption had arisen from the work of ahtz, referred to in the Paper, and Mr Park had illustrated a subtle point which limited application of Brahtz's argument to the very simplest cases. Dr Zienkiewicz, in his eresting recent Paper,[7] had shown that linearity was approximately compatible with ner, more or less reasonable, assumptions: that if the theory of homogeneous elastic ne strain was applicable, if the dam was not self-strained in any way, if the uplift eyed assumed distributions, and if the flow of pore water was strictly two-dimensional, en the variation of the material stress would be almost linear when Poisson's ratio = 0, hough it would approach Brahtz's prediction for values of Poisson's ratio higher than re likely to be applicable to concrete. It was a reflection of the complexity of the oblem that such assumptions were needed before a solution could be found.

Against that background Mr Serafim's careful experiments in Lisbon had been very lcome. Really valuable experiments into those questions were very difficult to make, d published data had been apt to provoke controversies as to the significance of the sults. Nevertheless, the linearity assumption was likely to be widely used for many ars to come, and both design engineers and research workers would be grateful for liable experimental results which would enable them to use it with more knowledge of its proximation to the truth.

Further References

. R. W. Carlson, "Permeability, pore pressure and uplift in gravity dams". Proc. Am. Soc. C.E., Paper No. 700. *See* discussion by J. L. Serafim, J. Power Divn, Proc. Am. Soc. C.E., No. PO1, p. 21 (Feb. 1956).

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. J. L. Serafim, "A subpressão nas barragens". Pubn No. 55, Lab. Nac. Engg Civ., Lisbon, 1954.

Paper No. 6089

The determination of the collapse loads of rigidly jointed frameworks with members in which the axial forces are large †

by

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Correspondence

Mr D. H. Clyde (Research student, Engineering Laboratory, University of Cambridg observed that the application of the simple methods of plastic analysis and design triangulated structures was not possible.[15] Such methods required a monotonical increasing relation between load and deflexion for validity, and the unloading effe examined by the Author violated that. On the grounds of prohibitive labour, the Auth ruled out a complicated non-linear analysis of the type performed on a digital comput by Foulkes[16] for a frame similar to those tested by the Author. The analysis suggeste as an alternative considered two states (i) wholly elastic and (ii) rigid-plastic in a theo derived for conditions of symmetry and single curvature. Plasticity occurred at poin which had also been the more highly stressed parts in the elastic range. For double-curv ture conditions, however, the early plastic deformation took place at points other tha those associated with the final collapse mechanism, especially when the slenderness rati were low $\left(< \pi \sqrt{\frac{E}{f_y}}\right)$. The mechanism considered by the Author was attained after unwrapping involving plastic deformation.[13, 14] It was thus to be expected that correl tion with experimental results would not be so good for the double-curvature tests Stevens and of Baker and Roderick as for the single-curvature tests of the Auth and of Baker and Roderick.

In order to derive an unloading line such as the Author's plastic-collapse line, a structu was examined at various states of deformation and the load necessary to maintain plast deformation was calculated. Plastic deformation occurred when the yield conditi was satisfied at sufficient places to form a mechanism. The method known as the pri ciple of virtual work or virtual displacements was a tool for writing out an equilibriu equation. A collapse mechanism was associated with that equation when the yie condition was satisfied at sufficient places (hinges) to form a mechanism. The elegan of the virtual-work approach lay in the fact that the imaginary mechanism used to wri the equilibrium equation was the same as the real one associated with the breakdov of that equation so that intermediate steps might be omitted. By neglecting rigid bod rotations CAA′ and CBB′ the Author formed equation (16) incorrectly but the corre form of the equilibrium equation from which equation (16) might be derived was used form equation (17). Fig. 9 had been redrawn (as Fig. 14a and Fig. 14b) with caa′ a cbb′ added. From Fig. 14b:

$$W \frac{d_c}{4} \cot \alpha = (M_P')_{AB} - \{(M_P')_{AB} - (M_P')_{AC}\}\frac{2d_c}{4h} \quad . \quad . \quad . \quad (16a)$$

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† Proc. Instn Civ. Engrs, Pt III, vol. 5, p. 213 (Apr. 1956).

[15] E. Chwalla, "Three contributions on the loading question of statically indetermina steel trusses". Pubn Int. Assocn Bridge & Struct. Engg, vol. 2 (1933–34), p. 96.

[16] J. D. Foulkes, "The behaviour of a stiff jointed truss". Brit. Weld. Res. Asso Report No. FE 1/44/54.

If d_c was very small compared with h:

$$\frac{Wd_c}{4}\cot\alpha = M_P' \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (16b)$$

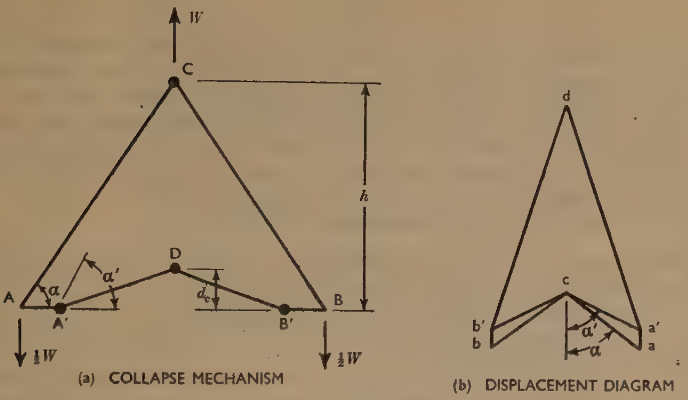

FIG. 14

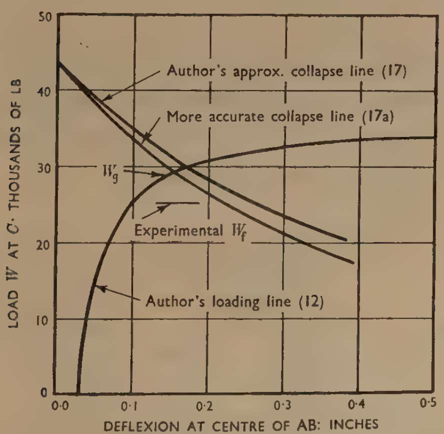

FIG. 15

n order to combine equation (16b) with equation (9) it was necessary to know the force in AB. Resolving forces at A (Fig. 8):

$$P_1 = \frac{W}{2}\cot\alpha \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (16c)$$

nce, from equations (16b), (16c), and (9):

$$W = 2\,P_Y\tan\alpha\left(\sqrt{\left(\frac{d_c}{d}\right)^2 + 1} - \frac{d_c}{d}\right) \quad . \quad . \quad . \quad . \quad (17a)$$

he alternative derivation was to consider A′D. There could be no vertical shear by

symmetry so the change from M_P' sagging at A′ to M_P' hogging at D was brought about by the horizontal shear of P_1. Hence:

$$P_1 = \frac{2M_{P'}}{d_c} \quad \text{. . . . . (16d)}$$

Equation (16b) might be obtained by combining equations (16c) and (16d). It should be noted that the solution thus derived on a kinematic basis and from the equilibrium of a portion of the frame required checking to see whether a bending-moment diagram could be derived from it to cover the whole frame without violating the yield condition. The solution given was not more than a few per cent different from that of the Author. It would, however, give a consistent reduction in W_g and had been plotted for the Author's worked example of frame 11 to give a 3% reduction (Fig. 15). The Author's estimate of 80% for the ratio of $W_f : W_g$ was thus slightly conservative.

Mr W. Merchant and **Mr C. A. Rashid** (Department of Building and Structural Engineering, College of Technology, Manchester) observed that the use of the intersection of the elastic stability line with the collapse mechanism line to determine an approximation to the failure load of a structure was not new; it had been so used at the College of Technology, Manchester, for several years.

The original suggestion from the Author was that for triangulated frames, the influence of secondary stresses on the elastic stability line could be neglected by comparison with the effect of initial curvatures and end eccentricities of members. Since Mr Merchant and Mr Rashid were interested in the general problem they were grateful to the Author for permission and assistance in carrying out a further analysis of his test results.

Secondary moments, if stability effects were neglected, were proportional to the applied load W and in that case gave rise to an inward central deflexion of the strut also proportional to W. Supposing that the elastic deflexion calculated without reference to stability effects was αW, then, allowing for stability effects $\delta_s \approx \dfrac{\alpha W}{1 - W/W_c}$ would be taken for the deflexion; that had to be added to the deflexion resulting from initial curvature and end eccentricities $\delta_e \approx \dfrac{\delta_0 + e'}{1 - \dfrac{W}{W_c}}$ as used by the Author.

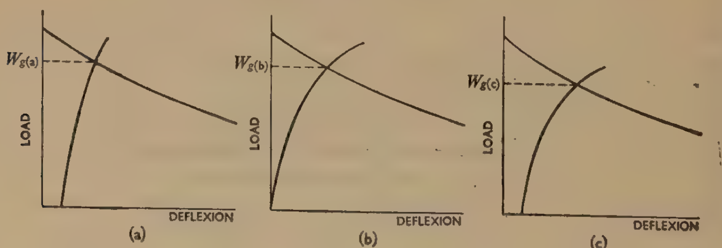

Fig. 16

Figs 16a, 16b, 16c showed three methods of obtaining W_g.
Fig. 16a showed the Author's method when the central deflexion taken was only δ_e.
Fig. 16b showed a similar method using only δ_s.
Fig. 16c showed a similar method using $\delta_s + \delta_e$.

To investigate the importance of δ_s, W_g had been evaluated by the three methods listed above and the results were given in Table 8. The Author seemed to assume that all moments arising from the end eccentricities were taken by the strut only whereas they

Frame No.	Author's* initial curvature $\delta_0 \times 10^3$	Author's* equivalent eccentricity $e \times 10^3$	Corrected equivalent eccentricity $e' \times 10^3$	Author's Y (central) $(\delta_0+e) \times 10^3$	Author's corrected Y (central) $(\delta_0+e') \times 10^3$	Author's W_g	Author's corrected $W_g(a)$	$W_g(b)$ secondary effect	$W_g(c)$ combined initial and secondary effects	W_y	W_c	W_f	$\frac{W_f}{W_g(a)}$	$\frac{W_f}{W_g(c)}$
1	−30	+ 3·5	+ 0·72	−26·5	−29·28	24,300	24,000	24,600	23,500	44,500	25,960	19,720	0·822	0·838
1′	−44	− 4·0	− 0·39	−48·0	−44·39	23,400	24,100	25,000	22,900	52,700	25,960	19,270	0·800	0·841
1″	−50	+29·5	+ 2·95	−20·5	−47·05	24,800	24,000	25,500	23,000	54,500	25,960	20,280	0·845	0·881
2	+12	−24·0	− 3·90	−12·0	+ 8·10	10,550	—	10,500	10,700	20,500	11,240	8,650	—	0·806
2″	+54	−59·0	− 9·88	− 5·0	+44·12	11,000	—	11,000	—	25,100	11,240	8,850	—	—
3	−43	− 1·0	− 0·10	−44·0	−43·1	33,600	34,250	38,000	32,700	46,900	44,570	24,900	0·726	0·761
5	−48	+12·0	+ 1·20	−36·0	−46·8	16,300	16,300	18,200	15,500	41,800	17,740	15,000	0·920	0·968
6	− 5	+42·0	+ 7·95	+37·0	+ 2·95	—	—	7,200	7,300	19,710	7,670	8,150	—	1·110
8	−35	− 7·0	− 0·77	−42·0	−35·77	42,300	43,000	45,000	41,000	46,900	94,400	35,130	0·816	0·856
9	−21	−18·5	− 3·07	−39·5	−24·07	11,200	11,350	11,600	11,000	11,850	43,400	9,250	0·815	0·841
10	−12	− 7·5	− 0·45	−19·5	−12·45	26,800	27,700	29,250	26,800	29,800	61,300	21,000	0·759	0·784
11	−20	−11·0	− 1·00	−31·0	−21·0	30,200	31,500	32,600	29,600	44,000	36,660	25,300	0·802	0·854
12	−20	+27·5	+ 5·89	+ 7·5	−14·11	—	14,200	13,800	13,000	20,300	16,250	12,380	0·871	0·952
13	−12	− 3·0	− 0·16	−15·0	−12·16	38,500	38,500	39,300	38,000	39,600	500,000	36,500	0·948	0·960
14	0	−41·0	− 4·08	−41·0	− 4·08	25,200	25,200	25,100	24,800	25,600	295,500	23,500	0·934	0·948
15	−11	+ 3·0	+ 0·82	− 8·0	−10·18	11,700	11,700	11,750	11,500	11,850	155,500	11,100	0·948	0·965
16	− 9	− 2·5	+ 0·38	−11·5	− 8·62	28,700	28,800	29,700	28,700	29,800	191,300	27,150	0·940	0·945
17	− 5	+ 6·0	+ 0·63	+ 1·0	− 4·37	19,000	19,000	19,200	18,800	19,200	115,000	16,600	0·874	0·878
18	0	−64·0	−11·95	−64·0	−11·95	8,780	7,600	8,600	7,400	8,860	52,800	7,030	0·924	0·948
19	−10	+16·0	+ 0·94	+ 6·0	− 9·06	—	38,200	39,500	36,250	42,900	52,300	27,200	0·712	0·750
20	− 7	+ 5·0	+ 0·57	− 2·0	− 6·43	25,400	25,000	25,000	23,200	27,750	32,100	20,200	0·808	0·871
21	0	−13·0	− 2·51	−13·0	− 2·51	10,900	11,700	10,200	9,860	12,790	13,200	9,070	0·778	0·920

Notes.—1. Frames 6, 12, and 19 in the Author's calculation failed on wrong side and now no longer failed in that manner.
2. 2 and 2″ are now failing in the wrong direction. They were the only two frames with positive initial curvature and hence there was a suspected error in the initial curvature.
3. In frames 2 and 6, initial positive Y_{central} had been compensated by secondary effect.
4. Frame 6, W_c was less than experimental failure load.

should have been shared between the members. The Author's values had been corrected to take that into account. It reduced the equivalent eccentricities considerably, especially when the tension members were much stiffer than the strut.

The ratios $\frac{W_g(a)}{W_y}$, $\frac{W_g(b)}{W_y}$, $\frac{W_g(c)}{W_y}$ were plotted against $\frac{W_y}{W_c}$ in Figs 17a, 17b, and 17c. Comparison of Figs 17a and 17b showed that for that series of tests, the inclusion of

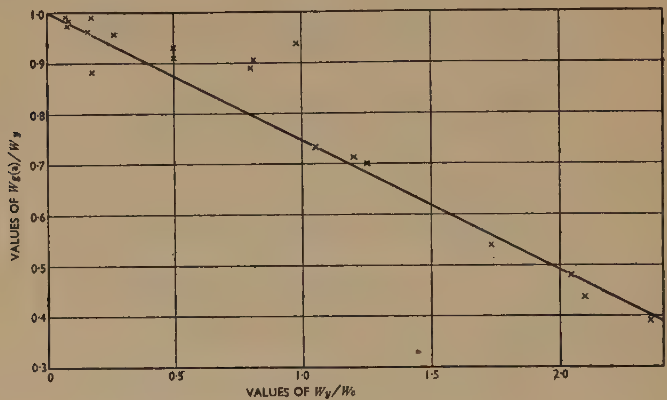

FIG. 17a

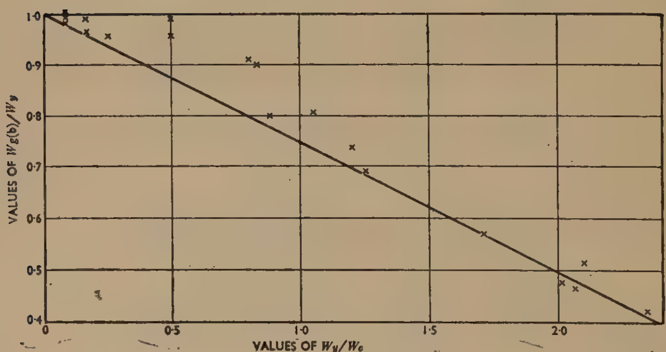

FIG. 17b

either δ_s or δ_e in the analysis would have a similar effect in determining values of W There was certainly no evidence that the secondary effects were negligible compared wit the initial curvature effects.

The point was not so obvious from a comparison of Figs 17a and 17c where the additic of δ_s to δ_e only appeared to cause a further slight reduction in W_g. The same compariso however, held between 17b and 17c and the apparent paradox was due to the non-line nature of the problem.

The method of manufacture of the series of frames appeared sufficiently consistent to ive rise to a fairly smooth relation between $\frac{W_g(\text{a})}{W_y}$ and $\frac{W_y}{W_c}$.

The similar smooth relation between $\frac{W_g(\text{b})}{W_y}$ and $\frac{W_y}{W_c}$ was not so surprising. The resultant $\frac{W_g(\text{c})}{W_y}$ and $\frac{W_y}{W_c}$ relation was also therefore fairly smooth.

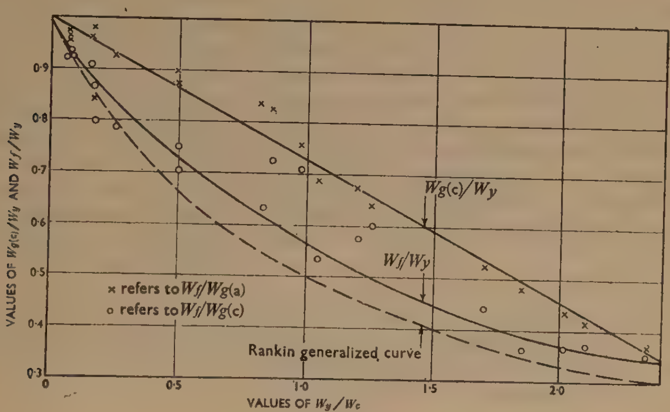

FIG. 17c

The revised ratio $\frac{W_f}{W_g(\text{a})}$ and the ratio $\frac{W_f}{W_g(\text{c})}$ were plotted against $\frac{W_y}{W_c}$ in Fig. 18. The pression of W_f below $W_g(\text{c})$ resulted from the combined effects of deterioration of bility and form factor. It appeared greater for values of $\frac{W_y}{W_c} \approx 1{\cdot}0$ than for either

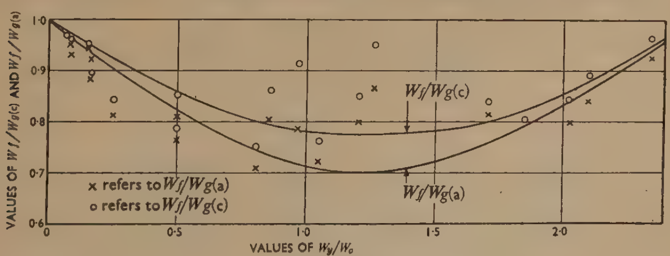

FIG. 18

der or stocky trusses. Since $\frac{W_g(\text{c})}{W_y}$ for these tests was fairly smooth it was worth ting $\frac{W_f}{W_y}$ directly; that had also been done in Fig. 17c. The generalized Rankine e $\frac{1}{W_f} = \frac{1}{W_y} + \frac{1}{W_c}$ was also shown for comparison.

should be remembered that that valuable series of tests was made on annealed frames

and accordingly there were no initial internal stresses. That difference from practical frames should be borne in mind.

Dr L. K. Stevens (Department of Civil Engineering, University of Melbourne) stated that the Paper showed that much more attention was now being given to the behaviour of rigid-jointed trusses. The mechanism by which collapse occurred was more fully understood, but theoretical methods of predicting collapse conditions still left much to be desired.

In Table 1 the minimum slenderness ratio was about 104. For the range shown an estimate of W_f equal to $0{\cdot}8\,W_g$ appeared to give reasonable results. However, as pointed out in Appendix II, W_f could be raised to $0{\cdot}95\,W_g$ for more stocky struts. The use of $0{\cdot}8\,W_g$ as an estimate of the collapse loads for rectangular sections might give conservative results, but an extension to non-rectangular sections would require considerable investigation before that attractively simple method could be accepted with confidence.

In Fig. 7 the presumably experimental points were shown as dots in the region where the experimental and predicted curves were in reasonable agreement. It was stated that for loads greater than 22,000 lb., "experimental deflexions are larger and the experimental curve becomes asymptotic to the plastic collapse line". Since the Author was using a loading mechanism which enabled controlled deformations to be applied, it was surprising that experimental points had been omitted in that region. It would be of interest to learn whether the broken line leading asymptotically to the plastic collapse line had been determined experimentally, or if it was based solely on theoretical grounds. In Fig. 11, the broken curves could be only an optimistic extrapolation of the experimental results, since the dead-weight and lever system, used by Baker and Roderick, did not allow the post-ultimate behaviour to be observed.

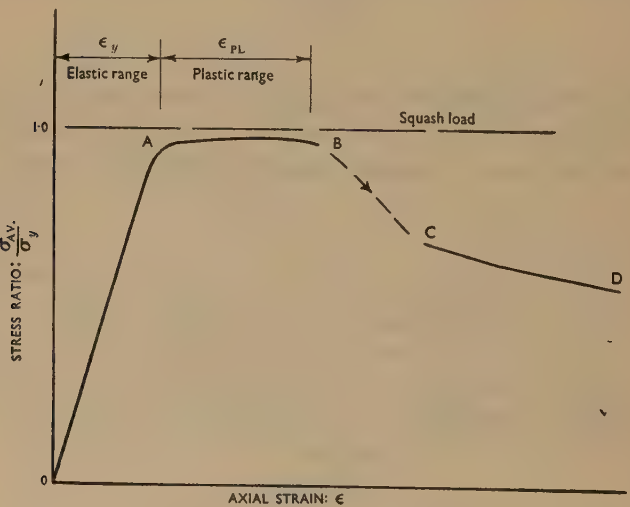

Fig. 19

The experimental verification of the behaviour after departure from the elastic stability line appeared rather doubtful. The verification of the theory would be more convincing if the Author were able to show experimental results in that region.

A further point on which the Author might care to give details was the large variation in yield point of the material used. In Table 7 that value ranged from 30,600 to 70,000 lb/sq. in. The shape of the stress/strain curve for those control specimens would also

of interest in determining the validity of applying the simple plastic theory to the ediction of the plastic-collapse line.

Further tests had recently been carried out by L. C. Schmidt on rigid-jointed trusses the University of Melbourne. The load-deformation relation of the critical strut s found to have the form shown in Fig. 19, provided that the slenderness ratio was s than about 110.

The line OA corresponded to the elastic stability line, and it seemed possible that CD ıld be part of some plastic collapse line. However, the behaviour between OA and CD s characterized by a definite flat-topped plastic range, AB. The length of this plastic ıge, ϵ_{PL}, was dependent upon the slenderness ratio of the critical strut, and to a lesser ent on the combined stiffness of adjoining restraining members. At B, an increase applied strain produced a sudden change of the strut configuration, and a new equiliium position was reached at C, with a reduced load. The slope and extent of the line depended upon the elasticity of the whole system of truss and testing machine.

The logical development in rigid-jointed statically indeterminate truss design appeared be the application of plastic-theory methods. For that to succeed, a knowledge of the gth of the plastic range, as well as the collapse load, was required. If the Author had results indicating the length and existence of a plastic range in his experiments, olication of them could be very helpful. Any explanation of it which could be made his theory would be welcomed.

The Author, in reply, said that Mr Clyde had given the correct equation for the plastic lapse line in equation (17a). The Author had derived equation (17) by neglecting cer- n small terms in the equation of virtual displacements. (In Fig. 9b it had been intended t a and b should be shown as a′ and b′.) By adding the points a and b to the displacent diagram Mr Clyde had allowed the loads $\frac{W}{2}$ at A and B to move through the additional tical distances a′a and b′b. Also the plastic hinges at A′ and B′ were rotated through les which were slightly less than those considered by the Author. If those small erences were both taken into account then the value of W was reduced as Mr Clyde had wn. It appeared that the simplest way of developing the plastic collapse line for a eral braced framework whose members were of rectangular cross-section was to sider the equilibrium condition as had been done in equations (16c) and (16d). The ıl load P_1 in the strut which failed was always proportional to the applied load W:

$$P_1 = KW \qquad (16e)$$

the general equation for the plastic collapse line became

$$W + \frac{P_Y}{K}\left(\sqrt{\left(\frac{d_c}{d}\right)^2 + 1} - \frac{d_c}{d}\right) \qquad (17b)$$

r Merchant and Mr Rashid had pointed out that the Author had erroneously assumed all moments arising from end eccentricities were taken by the strut. It was true they should be shared among the other members at the joint. A further refinement the inclusion of the effects of secondary moments. The effect of including those lifications in the Author's analysis was shown in Fig. 20. Frame 1″ had been chosen emonstrate those effects because it was the frame which had appeared to give least ement between the elastic stability line and the experimental results. The curve was that originally used by the Author assuming the equivalent strut shown in Table 1. allowing the tension members to take their share of the moments arising from end ntricities the equivalent strut was modified to that shown in the inset to Fig. 20 and elastic stability line was moved to ab_2. In the case of frame 1″ the constant 0·71 × 10⁶ in/lb. and if secondary moments were included the elastic stability line moved to ab_3. Fig. 20 showed that agreement with the measured deflexions was roved by modifying the analysis to include those two effects. The Author had shown[5] deflexions were quite sensitive to initial imperfections when slender members were
*

used. However, it was apparent from Table 8 that failure loads were comparatively insensitive to initial imperfections. It would therefore appear that beyond a certain degree of imperfection the collapse load of the structure was not materially affected by increases in those imperfections.

The struts of frames 2 and 2″ had both failed inwards. When applying the correct analysis to frame 2 the Author had found that an inwards failure of the strut was predicted

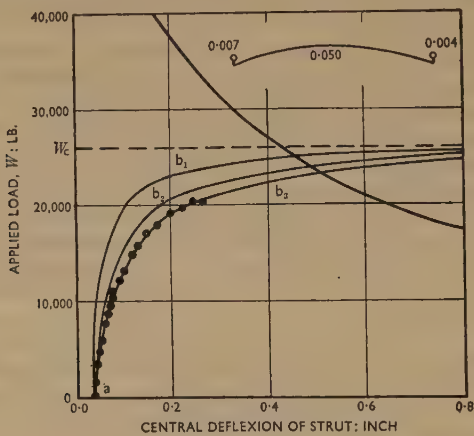

FIG. 20.—BEHAVIOUR OF FRAME 1″

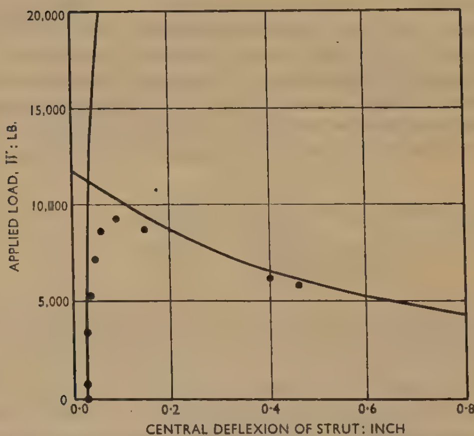

FIG. 21.—BEHAVIOUR OF FRAME 9

because of the high secondary moments ($\alpha = 2{\cdot}34 \times 10^{-6}$ in/lb.). However, when app ing the analysis to frame 2″ an outwards failure had been predicted. Dr Merchant a Mr Rashid suspected an error in the measured value of the initial curvature of the st Although the measurements would not be now checked the calculations had been and discrepancy was apparent. In an earlier elastic analysis[5] of that frame the Author l used a more exact analysis and had obtained good agreement with the experime

esults. A further examination of the results of that frame was probably desirable. The xplanation might lie in the curvature of the tension members. They have been neglected n the theory of the Paper under discussion but the measured values of curvature of the ies had been such that an inwards failure of the strut would be induced.

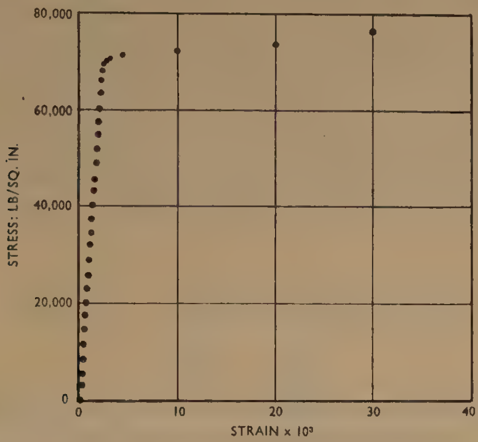

FIG. 22.—STRESS/STRAIN CURVE FOR A TENSILE SPECIMEN

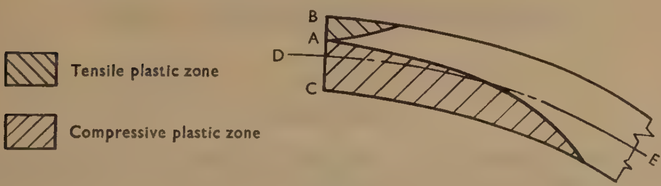

FIG. 23

Dr Stevens had mentioned that frames manufactured from bars of non-rectangular ction might give results which differed considerably from those made from bars of ctangular section. The Author had had further frames of a larger size made with a iew to investigating whether any appreciable variation did in fact occur. Results were ot yet available.

Collapse of the frames had been, in most cases, quite sudden, particularly in the case of ame 11. In such cases only the broken curve had been sketched in. The sudden collapse a frame was caused partly by the conversion of some of the elastic energy of the frame, ad meter, and testing machine into work done upon the plastic hinge (and also by the dden reduction in critical load when a plastic hinge formed at D). Equilibrium was not ached until the deflexions had become excessive and the frame had failed. The post-llapse behaviour of frame 9 had been observed at large deflexions at two loads and the sults were illustrated in Fig. 21. It would be seen that those points lay very close to the astic collapse line.

The material used for those frames was a bright drawn steel which must have had con-lerable work done on it in the drawing process and in its original state it had a high yield ress. Two types of heat treatment had been given. All of the first frames listed had en heat-treated for 1 hour at 550°C and that had not appreciably affected the yield ess. Because the strength of the stocky frames at that high yield stress would have

exceeded the capacity of the testing machine the frames were heat-treated at 900°C for 1 hour. That reduced the yield stress to 30,600 lb/sq. in. Fig. 22 showed the behaviour of one of the specimens treated at 550°C for 1 hour. It appeared that the material did strain without appreciable increase in stress as yielding occurred.

The Author had noticed a plastic range (Fig. 19) in his test results when stocky frames had been tested. It was thought that that behaviour would show more on the type of graph which Dr Stevens had plotted than the type plotted by the Author. It was suggested that the explanation was that the axial deformation of the strut which failed was greatly affected by the location of the plastic zones. Fig. 23 showed a strut on one side of a plastic hinge at section BC. The axial deformation was the integrated sum of the deformations of elements along the centre-line DE. In stocky struts A tended to lie nearer to B than was the case for more slender struts and the centre-line DE passed through a comparatively larger zone of plastically compressed material. The plastic range would then be more apparent on a load-axial deformation graph than on a load-lateral deformation graph.

Paper No. 6100

The canals of the Gezira canalization scheme and the design of the Goneid pump scheme in the Sudan †

by

Ian Stanley Gordon Matthews, M.A., M.I.C.E.

Correspondence

Mr H. F. Ayres (formerly of the Irrigation Department of the Egyptian Ministry of Public Works) observed that under the heading "Minor canals" the Author had stated that those canals were designed to store night flow not passed to the fields and had proceeded to describe how it was done.

Behind those simple statements lay the work of the late Mr A. D. Butcher, C.B.E., who was in the Irrigation Service of the Egyptian Government.

Shortly after the first world war, Mr Butcher had been appointed Director of the Hydraulic Research station at the Delta Barrage and carried out research into problems connected with irrigation in Egypt and the distribution of water in the Gezira area.

As a result of his investigations and especially his model experiments he had been able to advise the adoption of the system and devices referred to in the Paper.

Mr A. A. Middleton (Sir Murdoch MacDonald and Partners) stated that the Paper was useful in drawing attention to some considerations in detail design and construction of an irrigation scheme. He hoped that details would be given of designs and methods of construction adopted elsewhere for comparison.

As the Author had remarked, methods of design and construction adopted in the Sudan had been developed during many years. The spacing of the minor canals and field channels had, however, remained unchanged since the original scheme. It was a great tribute to the original designers that no modifications were desirable.

The Gezira irrigation scheme was one of the few large irrigation schemes where the water was supplied to the cultivators on demand. Agriculturally that was ideal but it

† Proc. Instn Civ. Engrs, Part III, vol. 5, p. 233 (Apr. 1956).

ded considerably to costs of constructing and operating the scheme. The canals had d to be designed to operate over a large range of discharges and the system of control d to allow frequent alteration in discharge. Accordingly, many regulators were cessary and the method of grouping canal heads shown in Fig. 2 had been developed.

The Author had described the system of night storage in the minor canals which had en adopted to avoid cultivators working during the night. That, like the supply of ter on demand, undoubtedly ensured that water supplied to crops was very carefully ntrolled and had been one of the factors contributing to the success of the scheme.

Since, during part of the year, the water contained a high proportion of very fine silt, ting of minor canals was inevitable, and a regular programme of clearance had to be rried out; the agricultural advantages, however, were such that the cost of silt clearance s considered to be worthwhile. However, a large proportion of the silt fell in the head ches and the banks would eventually become unmanageable. A feature of the ancient nal systems in Iraq was the vast banks near the heads of the branch canals. Often the nains of several channels could be seen where it had obviously been found more onomical, when the banks became large, to construct a new channel rather than clear existing one. Undoubtedly the time would come when the system of irrigation in the dan Gezira must be changed or a means of reducing the silt entering the canal would ve to be found.

Mr A. E. Griffin (Consultant) stated that coming about 27 years after the Papers of owde, Johnstone, and Russell [1–3] (which were written only 2 or 3 years after the eme had come into operation) he welcomed the Author's Paper and eagerly searched, t, for information on the performance of the night storage minor in relation to the perimental continuous flow minor and secondly, for some indication on progress made educating the cultivator to adopt normal (continuous-flow) watering methods. On th those points the Author was silent, which was unfortunate, having regard to the e of the Paper.

The Author, in his concluding paragraph, stated that he had "concentrated on the alization for which methods of design and construction have been developed over ny years in the Sudan and it is thought that this information may be of general use the design and construction of irrigation schemes". But in fact, an irrigation engineer s unlikely to adopt the Gezira scheme designs for general use though he might find m helpful if faced with a problem exactly similar to that in the early days of the zira scheme; that was a human problem entirely.

The original Gezira design was for normal watering. It was only after construction l been about three-quarters completed that it was decided (not by the Irrigation partment) to limit the watering period to daylight because the cultivator was new to gation and required much supervision, difficult to provide all round the clock. Although Irrigation Department had to accept that change and appreciated, to some extent, reason, they were aware of repercussions unless it was treated as temporary. In t, when a new agreement was being drafted between the Government and Concession-s, a clause was inserted to the effect that field watering would be carried out on normal s though Mr Griffin thought the latter resisted or partially resisted that. Moreover, Irrigation Department succeeded in putting in special areas where watering would continuous to train cultivators to become used to night watering and attendant siderations. The fact that no mention was made of those experiments in the Paper, him to wonder what headway was being made in that all-important matter.

The Author, in reply, observed that the unique conditions of the irrigation practice he Gezira scheme where the water was supplied to the cultivator on demand and only ing daylight hours was fully appreciated. He had not described that at length nor he discussed advantages and disadvantages of the "night-storage" system because he sidered that, despite that unique irrigation method, the sections of the canals and er-courses and the principle of design of water levels in the canals of the Gezira scheme

could be of general application. He felt that a lengthy discussion of the "night storage" aspects would have tended to confuse the issue.

The Author assured Mr Ayres that the contribution of Mr Butcher to the construction of the Gezira scheme was still fully appreciated in the Sudan and when water passed over the sills of the "night-storage" weirs the process was described as "Butchering", and the depth of water over a sill was known as the "Butchering depth".

In reply to Mr Griffin, the whole question of a "night-storage" as opposed to a "continuous-watering" system had been reviewed during the early stages of planning for the new Menagil extension, an area of about 800,000 acres lying to the south-west of the existing scheme. It had again been a question of weighing the agricultural advantages of "night storage" against economies in construction and maintenance of a "continuous-watering" system. The economy in construction of the "continuous-watering" scheme was estimated to be about £2/acre and economy in maintenance could not be expected to be greater than 2s or 3s/acre/year. Those economies resulted from the "continuous-watering" minor canals being spaced at double distances compared with the "night-storage" canals and accordingly only half the length of minor canals was required in the "continuous-watering" system. The working of the minor canals in the experimental areas of "continuous watering" had shown no appreciable reduction in silting and infection by floating weeds. That was because the terrain of the Sudan Gezira was so flat that even with "continuous-watering" minor canals it was not possible to provide sufficient water slope to preclude silting and the growth of floating weeds. Where land slopes allowed minor canals to be constructed with adequate water slopes the Author considered the case for a "continuous-watering" system would be greatly strengthened and that it should be adopted.

With the Gezira scheme the agriculturalists were able to counter the economies of "continuous watering" by a reduction in yield of between £2 and £3 per acre of cotton; that amounted to about 17s/acre/year for a one-to-three rotation. It had therefore been decided in 1954 despite the engineering objections to the "night-storage" system that the better service it provided to the agriculturalists justified the additional expenditure in first cost and maintenance and that it should be adopted for the Menagil extension.

With regard to silting of head reaches of the minor canals and the possibility of a radical change in the system of irrigation because of excessive spoil banks, as raised by Mr Middleton, that had already been a maintenance problem; in some canals the spoil banks had already encroached on the area of cultivation in the head reaches. The Author suggested that tractor-drawn earth-moving plant could be used to move the excavated spoil from the head reaches of the canals. That would not, however, provide a permanent solution and if the present system was to continue some form of silt extractor to remove the fine suspended silt while the water was still in the main canal would be required.

CORRIGENDA

Proceedings, Part III, August 1956, Paper No. 6111 (Karmalsky and Korner):

536, Fig. 1. For $h \,.\, p_0$ read $h \,.\, p_0'$

For $y_0 = B$ read $y_0' = B$

537, equation (9). For x_y read xy

539, equation for D_2. In the denominator of the third term, for $6P_2$ read $6P_1$.

544, line 21. For $P_h = 0$ read $p_h = 0$.

551, Fig. 7. *For* 596 lb/sq. ft *read* 5,960 lb/sq. ft.

554, line 13. The vinculum should extend over both parentheses.

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